

PRIORITY LANES ON URBAN FREEWAYS:
SOME OPERATIONAL CONSIDERATIONS

By

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To my wife, Maryanne Gillis Culpepper

*"Follow your own bent,
no matter what people say."*

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PRIORITY LANES ON URBAN FREEWAYS:
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Since its introduction in the late nineteenth century, the automobile has played a key role in shaping the current urban form and way of life in the United States. In recent decades, the emergence of the major problems of congestion, pollution, energy consumption, and environmental deterioration have pointed out the need to make more efficient use of current transportation resources. One approach to this problem which is now being investigated is encouraging travel in high-occupancy vehicles by providing positive incentives in the form of preferential treatment. A primary means of promoting this type travel is the reservation of freeway lanes for the exclusive use of high-occupancy vehicles during hours of peak demand. This dissertation is concerned with the development of techniques for investigating the optimal control and operation of these priority lane systems.

One of the major control parameters associated with this type priority treatment is the definition of a "high-occupancy" vehicle. This work develops a methodology for establishing the minimum number of occupants which should be required for qualification as a priority vehicle for a given reserved lane system. The proposed technique is a mathematical optimization model, based on linear programming, which considers the total system demand, the operating characteristics of the facility, and the desired degree of priority, and minimizes the total travel time for the system. This model is validated, various applications are discussed, and some general guidelines are developed.

Another important consideration in the operation of these systems is the manner in which reserved lane access and egress is to be provided. A methodology for investigating the priority lane entry/exit strategy is proposed, again based on a mathematical optimization technique. This model, a network flow analysis procedure, considers the system operating characteristics and the priority and nonpriority demands in the development of an entry/exit strategy which will minimize the total hours of travel within the system. This model is validated, a variety of applications are discussed, and some general observations relating to priority lane entry/exit are presented.

These techniques are applied to an existing priority lane system operating in Miami, Florida. Reasonable results were obtained, and the application demonstrated that the models are viable analysis tools.

CHAPTER 1

INTRODUCTION

This dissertation presents methodologies for determining optimal control parameters and operating strategies for reserved bus and car pool lanes on urban freeways.

Background and Rationale

America was introduced to the automobile in the late nineteenth century. By the year 1900 there were approximately 8,000 privately owned automobiles in operation on a total of 2,300,000 miles of roadway [Motor Vehicle Manufacturers Association (MVMA), 1976]. As the country grew and mass production developed, the dependence of the American people on the automobile also grew. It is estimated that in 1976, over 106 million automobiles were registered in this country. In the same year, approximately 1.6 trillion passenger miles were traveled on some 3,816,000 miles of roadway [MVMA, 1976].

During this period, this country also experienced a shift from an agricultural economy to an industrial economy. This produced a migration from the sparsely settled farmland areas to the more densely populated urban areas. It is now estimated that more than 50% of the U.S. population lives in suburban areas. This suburban dwelling pattern has increased the dependence on the automobile. In 1976, over 77% of the employed American public were dependent on the private

automobile for the daily travel to and from their place of employment. This home-to-work trip accounted for 31.9% of the total person-trips and 33.7% of the passenger-miles of travel in 1976 [MVMA, 1976]. In raw numbers, this means that 37,102,997,860 person-trips covering 357,626,566,400 vehicle-miles were required to transport the American work force to and from their places of employment.

In order to meet this steadily increasing load on the roadways, a number of methods have been used. Initially, the approach was an upgrading (surfacing) of the existing roadways. Between the years 1900 and 1976, the total roadway mileage increased only 65% (2.3 million miles to 3.8 million miles). However, during this same period, the percentage of paved roadway was increased from 8% to over 80% of the total mileage [MVMA, 1976]. Recognizing the need for even better highway facilities, a nationwide system of interstate and defense highways was conceived in 1944. This system of high-speed, limited-access roadways was 88% complete in 1975 with a total designated mileage of 42,500. The concept of high-speed facilities has been adopted in most metropolitan areas, and local cross-town expressways or freeways now serve a large proportion of the urban area travel.

As can be witnessed in any urban area, these methods based on providing more and more vehicular capacity have been unable to keep pace with increasing demand. Traffic congestion is now considered a "way of life" in many areas. Additionally, environmental and energy considerations have detracted from these alternatives during the past two decades. The American public is no longer willing to devote large portions of the land or energy resources to roadways. As a result of this increased

environmental concern and the recent energy shortages in the world, alternative methods for meeting the country's transportation requirements are now being explored.

One of the more obvious alternatives is mass transit. This mode of transportation has been available, in various forms, since the days of the stagecoach. Recently, the development or expansion of conventional rapid rail and fixed-route bus systems has been utilized in several localities. Additionally, experimentation with the concept of demand-responsive systems in several forms has been in progress. This activity has been successful to varying degrees and future promise is evident in this approach. However, it should be noted that one of the primary drawbacks to these systems is the relatively low population density in this country. In European countries where the average population densities range from 200 to over 800 persons per square mile, this approach has been very successful. The average density in the U.S. is 57 persons per square mile [MVMA, 1976]. In this lower density situation, mass transit simply cannot be made as convenient for the American public as for their European counterparts.

A second alternative now being explored is making more efficient use of the automobile by increasing the average occupancy level. This approach has the effect of meeting the total person demand while reducing both the vehicular demand and the overall energy consumption. The primary target area for this effort is the peak-hour, home-to-work travel. This travel accounts for 33.7% of the yearly person travel and is made during the combined daily peak-periods, covering only 4 to 6 hours per day. Also, the average automobile occupancy during peak-

periods is only 1.4 persons per vehicle, somewhat lower than the overall average of 1.6 persons per vehicle [MVMA, 1976].

As a means of achieving this increased automobile efficiency and, at the same time, encouraging the use of public transportation, consideration is being given to providing preferential treatment for high-occupancy vehicles in the urban corridors. This is not a new or revolutionary concept since transit priority in the form of exclusive rights-of-way was first introduced in Chicago in 1939 [United States Department of Transportation (USDOT), 1975]. More recently, the provision of preferential or priority treatments on freeways has been given serious consideration. A summary of the advantages and disadvantages of current techniques in this area is presented in Table 1.1.

The use of normal-flow reserved lanes, which forms the subject of this investigation, offers potential benefits in the reduction of both vehicular demand and energy consumption, while meeting the total person demand. The person capacity of a single freeway lane is approximately 2,800 persons per hour (2,000 vehicles per hour at an average occupancy of 1.4 persons per vehicle). The same lane could carry 8,000 persons per hour if used only by car pools of 4 persons, or 25,800 persons per hour as a bus-only lane. As to the energy consumption, an increase in the average peak-hour occupancy of 1.4 persons per vehicle to the overall average of 1.6 persons per vehicle would save an estimated 2.3 billion gallons of gasoline per year. More realistically, an increase in the average occupancy to 1.5 persons per vehicle, which has been achieved in 2 priority freeway lane projects [USDOT, 1975], would save approximately 1.2 billion gallons of gasoline each year.

Table 1.1 SUMMARY OF THE CHARACTERISTICS OF VARIOUS FREEWAY PRIORITY TECHNIQUES

Technique	Examples	Advantages	Disadvantages
Construction of exclusive rights-of-way	Shirley Busway, Washington, D.C. San Bernardino Busway, Los Angeles, CA Pittsburg PATways, Pittsburg, PA	1) Priority vehicles can operate at high speeds 2) Efficiency of existing highway not reduced 3) Operating costs are low 4) Easily enforceable	1) Major construction takes long implementation time 2) Capital costs are relatively high 3) Community opposition may develop
Normal-flow reserved lanes	San Francisco-Oakland Bay Bridge, CA Marin County, San Francisco, CA Moanalua Freeway, Honolulu, HI	1) Involves minimum construction 2) Can be rapidly implemented	1) Weaving problems 2) Takes lanes away from peak direction 3) Enforcement may be more difficult
Contra-flow reserved lanes	I-495 Lincoln Tunnel, NY/NJ Long Island Expressway, NY Southeast Expressway, Boston, MA Marin County, San Francisco, CA	1) Capacity in peak direction is increased 2) Capital outlay is low 3) Such lanes are rapidly implemented	1) Should not be implemented if congestion would develop in off-peak direction 2) Operating costs are relatively large 3) Bus speeds are relatively low
Preferential treatment at freeway ramps	I-5 Blue Streak, Seattle, WA Los Angeles area freeway ramps, CA I-35W, Minneapolis, MN	1) Low construction costs 2) Causes minimum delay to nonusers 3) Problem of freeway underutilization is avoided 4) Speed difference is diminished, so safety is enhanced	1) Violations may be high 2) May not always be physically possible 3) Priority vehicles subject to delays once they are on freeways during days with accidents or incidents

Source: Priority Techniques for High Occupancy Vehicles, U.S. Department of Transportation, 1975.

Statement of the Problem

In the area of designing and evaluating priority treatments for high-occupancy vehicles, a substantial amount of work has been done. This work has provided techniques for designing preferential treatments for freeway ramps in terms of the priority cut-off level or car pool definition and control strategies for nonpriority demand, and for simulating the operation of priority ramp systems, reserved normal or contra-flow freeway lanes or exclusive rights-of-way. However, the current state of the art does not directly address the design and operational criteria for reserved freeway lanes. The intent of this effort then is directed toward the development of methodologies to investigate operational design of these priority lanes on urban freeways.

Scope of the Study

Two of the major operational considerations for a reserved high-occupancy vehicle lane on an urban freeway are (1) the definition of "high-occupancy" and (2) the provision of entry/exit points for the lane. The primary focus of this study will be the development of techniques which can be utilized to define a high-occupancy vehicle or car pool and to determine the locations at which priority access and egress should be provided. Additionally, it will be demonstrated that these methodologies can be used to investigate other aspects of priority lane operations, such as the geographical limits and hours of operation.

Organization

The presentation of the methodology and results of the project is contained in the following chapters. A survey of current methodologies for priority treatment investigations is provided in Chapter 2. This is followed by the development of the proposed techniques for establishing a car pool definition and identifying an optimal entry/exit strategy in Chapters 3 and 5, respectively. Chapters 4 and 6 contain demonstrations of the validity of the proposed techniques, as well as discussion of their areas of application and some general guidelines or observations.

A case study application of the proposed techniques to an actual reserved lane system in operation on I-95 in Miami, Florida, is presented in Chapter 7. Conclusions based on this investigation and suggestions for future research are contained in Chapter 8.

CHAPTER 2

SURVEY OF THE LITERATURE

Introduction

Classification of This Investigation

The work that was carried out in support of this dissertation can be properly classified as an application of systems analysis techniques to an investigation of traffic flow. Specifically, the work presented in this report is directed toward the development of techniques for determining optimal control and operating strategies for reserved lanes on urban freeways.

Scope of the Review

Inasmuch as a review of all previous efforts in the area of traffic-flow theory and systems analysis is well beyond the scope of this work and would contribute little to the final product, this review will be limited to those previous efforts in the area of modeling freeway control systems, particularly reserved lane operations. In the discussion to follow, primary consideration will be given to the nature and application of previously developed freeway control system models which are related to this investigation. Presentations of the various techniques and methodologies adopted in this work will be made as the models are developed in subsequent chapters.

For additional information related to freeway control systems, reference can be made to previously compiled state-of-the-art documents. These include a comprehensive survey of current freeway surveillance and control techniques by Everall [1972], and detailed guidelines for design and operation of freeway ramp control systems by Masher et al. [1975].

Previous Research

Early Applications

The idea of applying systems analysis techniques in the area of traffic engineering is not a recent development. This concept was first suggested by Lewis [1954] and utilized by Edie [1954] in determining the number of toll booths required on the George Washington Bridge in New York. The use of optimization techniques in developing a minimal travel time assignment of vehicles to a traffic network was first demonstrated by Charnes and Cooper [1959]. This model was later used by Pinnell and Satterly [1962] to determine the optimal operation of a freeway with a continuous frontage road.

Freeway Models

The use of systems analysis in investigations on freeway on-ramp controls was first demonstrated by Wattleworth [1962]. This model was based on a linear programming approach and was designed to determine the optimal metering rates for a series of ramps. This basic methodology has been adopted in a variety of subsequent efforts, including those of Goolsby et al. [1969], Messer [1969], Brewer et al. [1969], Wang and May [1973a], and Ovaici and May [1975].

A general freeway operations evaluation model based on a simulation approach was proposed by Makigami et al. [1970]. This model, known as the FREEQ model, was applicable to investigations of the operating characteristics of directional freeways and has been applied successfully by Allen and May [1970] and Stock et al. [1971]. This model was later refined by Blankenhorn and May [1972] and then again by Stock et al. [1973]. As a result of these refinements, the FREEQ3 model was developed. This model, still a simulation-based technique, now evaluated directional freeway operations with consideration given to implementation of ramp control strategies, such as metering. This final model formulation was used successfully in an investigation of the East Shore Freeway in the San Francisco Bay Area [May, 1974].

A methodology for designing freeway ramp control strategies based on the FREEQ3 model was proposed by Eldor and May [1973]. This model, known as FREEQ3D, was, in essence, a search process based on iterative application of the FREEQ3 model. This procedure was subsequently converted to a deterministic optimization model by Wang and May [1973a]. The iterative process in FREEQ3D was replaced by a linear programming technique similar to the one proposed by Wattleworth. The resulting model, called FRELQ3C, used the optimization technique to determine the optimal metering rates for each freeway ramp, and then simulated operations before and after control as in previous models.

Subsequent to the initiation of a program to reduce traffic congestion by providing preferential treatment for buses and multi-passenger vehicles, a number of priority treatment models have been

developed. The first of these was proposed by May [1968]. This rudimentary model was designed to simulate the operation of an exclusive bus lane on a freeway and was based on the assumption of constant peak-period demand and a simple Greenshields flow submodel. This model was later refined by Stock [1969] to include consideration of time-varying demands and a variety of speed-flow submodels. The name EXBUS was adopted for this refined model.

In order to consider exclusive lanes for both buses and car pools, Sparks and May [1970] proposed another step in the evolution of the EXBUS model. This third-generation model, still known as EXBUS, simulated the operation of a freeway lane reserved for the joint use of buses and car pools. This final version of the EXBUS model has been used in priority lane feasibility studies for the San Francisco-Oakland Bay Bridge [Martin, 1970] and for a segment of I-90 in Cleveland, Ohio [Capelle et al., 1972].

As a result of the application of the EXBUS model in these feasibility studies, some weaknesses in the model structure and application procedure were identified. Specifically, the simulation model lacked the capability to consider temporal or spatial variations in the demand and/or capacity and manual interfacing of the priority lane simulation, and a simulation of the normal lanes was required to determine the operation of the total system. These weaknesses prompted the development of the PRIIFRE model by Minister et al. [1973]. This model combined the philosophy of the EXBUS model and the more realistic approach of the FREEQ model, and allowed simulation of a directional freeway operating with one or more lanes reserved for the use of buses

and car pools. The PRIFRE model was developed primarily to simulate one-way "normal" priority lane operations, i.e., the reserved lane(s) on the same side of the median as the nonreserved lanes. However, with manual interfacing, it can be applied to contraflow on reversible lanes, separate priority roadways, freeway design alternatives, and evaluation of ramp control strategies.

A second consequence of this program to provide preferential treatment for high-occupancy vehicles was the need to consider priority access at freeway on-ramps. In response to this, the FREQ3C model was modified by Ovaici et al. [1975]. The modified model, known as FREQ3CP, uses a deterministic optimization technique to determine the priority cut-off level for ramp priority and the optimum metering rates for nonpriority vehicles at each on-ramp. Additionally, the model simulates the system operation with and without the provision of priority access. In the simulation process, an on-freeway priority lane may also be considered.

Corridor Models

It is also possible to evaluate freeway control strategies as a part of the freeway corridor operations. One model which has been developed for simulation of corridor operations is the SCOT model proposed by Lieberman [1971]. This model is based on the prototype Urban Traffic Control Simulator (UTCS-1) model developed by Bruggeman et al. [1971] for network flows and on another model, DAFT, proposed by Lieberman [1970] for the dynamic assignment of freeway corridor traffic. In this assignment and simulation process it is possible to consider

freeway control strategies in the form of ramp metering or closure [Lieberman, 1971].

Another corridor model, CORQ, has been proposed by Yagar [1975]. This model was designed to predict the self-assignment of time-varying demand in a freeway corridor. The basic structure of this model is an iterative simulation process in which the flow assignment is determined. With appropriate application, this model can be used to evaluate or design normal ramp control strategies, however, priority treatments cannot be considered.

Another possibility for considering various freeway control strategies is in the context of a full network model. The primary development in this area is the Urban Traffic Control Simulator (UTCS) model mentioned previously. The prototype for this network simulation model was proposed by Bruggeman et al. [1971], and additional refinements were suggested by Lieberman et al. [1972]. The UTCS model is a microscopic simulation model designed as an evaluative tool for urban traffic control policies. As such, the model might be used to evaluate alternative freeway control strategies, but evaluation or design of priority treatments would be outside its realm of applicability.

Relevance to This Effort

As has been demonstrated, previous research in the area of freeway control systems has produced a number of models for (1) investigating normal freeway operations, (2) simulating and designing freeway ramp control strategies with or without priority considerations, (3) simulating freeways with reserved lane operations, and (4) consider-

ing freeway control strategies in relation to their corridor or network effects.

However, these efforts have not directly addressed the determination of optimal reserved lane control parameters and operating strategies, which is the subject of this investigation. Therefore, the contributions of previous research to this particular study are limited to (1) establishing an overall philosophy for the development of traffic flow models, (2) identifying acceptable procedures for use in modeling efforts, and (3) demonstrating the validity of applying systems analysis techniques in traffic engineering.

Summary

A summary of the characteristics of these previously developed freeway control system analysis models is presented in Table 2.1. The models to be developed in the remainder of this work have been included in this summary table in order to demonstrate the manner in which they extend the range of currently available techniques.

Table 2.1 SUMMARY OF PREVIOUS AND PROPOSED MODELS

Name	Model		Area of Application	Basic Technique	Comments
	Developed By				
	Wattleworth		Freeway entrance ramp control systems	Linear programming	First freeway ramp control model to be proposed
FREEQ	Makigami, Woodie and May		Basic freeway operations	Simulation	
FREQ2	Blankenhorn and May		Basic freeway operations	Simulation	Refinement of the FREEQ model
FREQ3	Stock, Blankenhorn and May		Basic freeway operations with ramp controls	Simulation	Additional refinement of the FREEQ model
FREQ3D	Eldor and May		Design of freeway ramp control systems	Simulation	Iterative application of the FREQ3 model
FREQ3C	Wang and May		Design of freeway ramp control systems	Simulation and linear programming	Refinement of the FREQ3D model
	May		Freeway operations with exclusive bus lanes	Simulation	Rudimentary model
EXBUS	Sparks and May		Freeway operations with exclusive bus lanes	Simulation	Refinement of the May model

Table 2.1 - continued

Model		Area of Application	Basic Technique	Comments
Name	Developed By			
EXBUS	Sparks and May	Freeway operations with full priority lanes	Simulation	Additional refinement of the May model
PRIFRE	Minister, Lew, Ovaici and May	Freeway operations with full priority lanes	Simulation	Combination of the FREEQ and EXBUS models
FREQ3CP	Ovaici, May, Teal and Ray	Design of freeway ramp control systems with priority treatment	Simulation and linear programming	Also simulates priority freeway lane operation
SCOT	Lieberman	Freeway corridor operations	Simulation and dynamic assignment	Considers freeway ramp controls; Based on UTCS-1 and DAFT models
CORQ	Yagar	Freeway corridor operations	Simulation and dynamic assignment	
UTCS-1	Bruggeman, Lieberman and Worral	Transportation network operations	Simulation	
CARPOOL [*]	Culpepper	Priority lane operations on freeways	Assignment and linear programming	Determines optimal car pool definition
STRATEGY [*]	Culpepper	Priority lane operations on freeways	Network optimization	Determines optimal priority lane entry/exit strategy

* Proposed in this work

CHAPTER 3

DEVELOPMENT OF A CAR POOL DEFINITION MODEL

Introduction

Objectives

One of the major control parameters associated with any priority treatment technique is the definition of a priority vehicle. In the case of transit priority treatments, this definition is simple and straightforward. The transit vehicles, i.e., buses, are the only component of the traffic demand to be given priority status. However, in the case of a high-occupancy vehicle (HOV) priority system, the definition of a priority vehicle takes on a new dimension. For these treatments, the level of passenger occupancy which is to be considered "high-occupancy" must be determined. Although this decision is often influenced by convention or social and political considerations, it should ideally be based on an application of sound engineering principles in each particular situation.

In this chapter, an examination of the engineering considerations involved in making this determination will be presented. As a result of these considerations, a methodology for investigating the level of occupancy which should constitute "high-occupancy" for a particular HOV priority treatment will be developed. This development

will address both the framework for the investigation and the mathematical tools which will be utilized.

Description of the Problem

As is the case with any proposed methodology, certain limitations with respect to the range of applicability must be imposed on the problem at hand. In this case, the techniques to be developed will be restricted to investigations of HOV priority lanes on urban freeways. The motivation for this is twofold. First, it is unlikely that the need for implementing priority treatments will develop outside the populous urban areas. Second, previous research [Ovaici et al., 1975] has yielded acceptable methodologies for defining "high-occupancy" for priority entry systems.

Additionally, it will be assumed that the decision to implement reserved lanes treatment rather than a priority entry system has been made. Contrasts or comparisons of these alternatives will not be considered as a part of this effort. It will also be assumed that the number of lanes to be reserved for priority traffic has been predetermined. However, as will be discussed in subsequent chapters, this car pool definition methodology can be readily adapted for making this determination as well as investigating additional reserved lane concepts.

Therefore, considering these limitations and assumptions, the methodology to be developed will be a technique for determining the level of occupancy which should constitute priority status for HOV priority lanes on urban freeways. This minimum level of passenger occupancy or priority cutoff level will then be the car pool definition for the reserved lane system.

Method of Analysis

A primary consideration in the development of any analysis methodology such as the one proposed here should be the work of previous reserchers. As was discussed earlier, there is little in the current literature which might be used as a basis for this investigation. The work that has been done is limited in scope, oriented toward other prime objectives, or only generically related to this effort. However, the research that is documented does provide a general background for this investigation in the form of philosophical guidelines and acceptable techniques for modeling traffic flow and control systems.

It is then possible to consider the basic structure of a car pool definition model. When evaluating candidate techniques to be used as a basis for any mathematical model, certain desirable characteristics should be considered. For this particular effort, the following criteria were used:

1. The model should adequately reflect the characteristics of the physical system.
2. The model should address a reasonably broad range of alternative situations.
3. The model should be adaptable for investigations of special conditions.
4. The model should, to the extent possible, be based on currently acceptable techniques.

The candidate techniques which were identified for this effort include: (1) analytical models, (2) simulation models, and (3) optimization models. Analytical modeling would, in this case, consist of developing mathematical expressions which describe the various relationships of traffic flow and combining them to develop an expression

defining the car pool requirements in terms of the traffic flow parameters. Development of a simulation model would require formulation of a mathematical framework which would describe the traffic flow interactions encountered in an HOV priority treatment. This model would then be utilized to evaluate the system operation for different combinations of the control parameters. Optimization techniques also require development of mathematical descriptions of the traffic flow relationships and system characteristics, but would be used to determine the manner in which the system should be controlled to achieve "optimal" operation.

Detailed examination of the potential value of an analytical technique in this case quickly demonstrates several weaknesses in the approach. Primarily, there is a tendency for this technique to become unwieldy or unsolvable for complex situations. Although an analytical model can be used successfully with artificially simple representations of the traffic flow interactions, the degree of simplification required to achieve a manageable model would be costly in terms of maintaining a realistic representation of the physical system. Another significant weakness in this approach is that the resulting expressions are generally more narrow in their range of application than is desirable for this effort. Finally, the resulting expressions are not readily adaptable for use under special conditions.

Simulation techniques overcome the limitations of the analytical techniques to a large degree. These models, if properly developed, can realistically represent the physical system operations and are generally quite flexible in application. Some difficulties may be

encountered in adapting a model of this type for investigations of special conditions, but, as a general rule, this can be accomplished more readily than would be the case with analytical models. For the purposes of this work, the simulation approach does have one major drawback. The basic intent of any simulation model is to provide information on how a system will operate under fixed conditions rather than determining the conditions under which optimal performance will be achieved. This technique can be utilized to evaluate alternatives in an exhaustive search process which will ultimately define the best or optimal conditions for system operation. It is this lack of deterministic results that has led many operations research analysts to refer to simulation as a "method of last resort" [Wagner, 1975, p. 907]. It should be pointed out, however, that simulation techniques do have their rightful place as a method of evaluating system operations and have been utilized quite successfully in modeling various priority treatment techniques [Ovaici et al., 1975; Minister et al., 1973].

The last method to be considered is that of optimization models. This approach is one in which the ultimate goal is the optimization, i.e., maximization or minimization, of a numerical function of a set of variables which are subject to a number of constraints [Hadley, 1963, p. 1]. This class of techniques exhibits many of the strengths of simulation, such as realistic representation of the physical system, formulation for broad-range applications, and adaptability for special conditions. Additionally, this approach is directed toward a decision-making process as opposed to an evaluation of system operations.

Specifically, these techniques deal with the optimal allocation of limited resources to meet given objectives. This approach has been utilized in determining optimum control parameters for several traffic control systems since the early 1960s [Wattleworth, 1962].

The preceding considerations indicate that an optimization technique would be the most direct approach, and is, therefore, the approach that will be taken in the development of a car pool definition model.

Development of the Basic Model Structure

Description of the Physical System

The first step in developing any mathematical model must be the definition of the system which is to be modeled. In the case of a car pool definition model for an HOV priority lane on an urban freeway, the basic component of the physical system is the freeway. Specifically, it is that section of the freeway in which priority treatment is to be provided. This freeway section is in reality composed of two distinct subsections, the lane(s) reserved for priority traffic and the lanes remaining for nonpriority traffic. Although these subsections operate concurrently in time and space, they do possess distinct operating characteristics, such as capacity and speed-flow relationships. If the utilization of roadway capacity by individual vehicles is viewed as a basic supply-and-demand situation, these subsections would represent the supply portion of the system. This "supply" of roadway capacity is then the resource which is to be allocated.

The demand portion of this relationship is represented by the individual vehicles desiring to use this section of freeway. This

"demand" can be viewed as the total number of vehicles which use the roadway or as a stratified demand consisting of vehicles with a single occupant, vehicles with two occupants, etc. For the purposes of this effort, the latter viewpoint will be taken for reasons to be discussed later.

The physical system to be modeled is then a section of freeway with both reserved and nonreserved lanes, the available capacity of which are to be allocated to the vehicular demand in such a manner as to optimize the system operation. The objective of the model is to determine the level of occupancy which should be required for priority status such that optimal operation is realized. This system is shown graphically in Figure 3.1.

General Modeling Considerations

One common element of all optimization models is the objective function. This is some mathematical function of a set of variables, known as decision variables, the value of which is the quantity to be maximized or minimized. This expression is normally considered to be the "cost" associated with operating a particular system in a certain manner. In the area of traffic flow analysis, a wide variety of objective functions or figures of merit have been proposed and utilized. Among these are such measures as total travel time, total system input rate, total system output rate, vehicle-miles of travel, and passenger-miles of travel.

In considering the figure of merit to be utilized as an objective function for this model, it was felt that the selected criterion

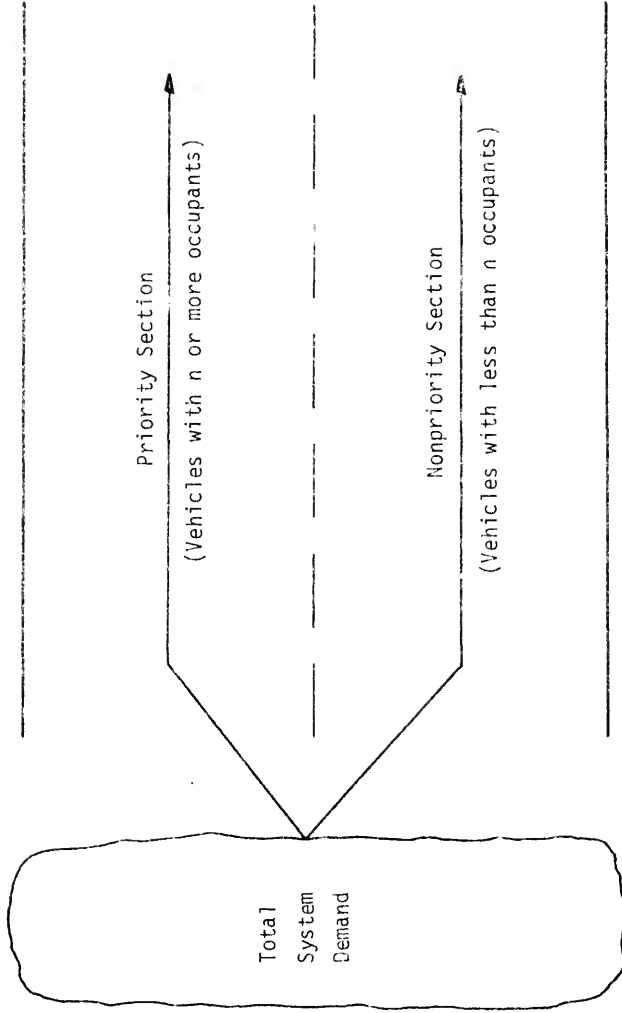


Figure 3.1 SCHEMATIC MODEL OF A HIGH-OCCUPANCY VEHICLE PRIORITY LANE SYSTEM

should directly reflect the fact that the overall intent of priority treatment is to reduce the trip time for high-occupancy vehicles. Additionally, the selected figure of merit must not be independent of the control parameter under consideration, nor can it be artificially optimized at the expense of overall system operation.

Looking first at the two latter considerations, such measures as system input and output rates and total miles of travel would be independent of the control parameter inasmuch as the total demand is fixed and no diversion will be considered. Previous investigations have shown that under certain conditions, specifically ramp control without diversion, maximization of system output may be equivalent to minimization of total travel time. However, since ramp controls will not be considered in this model, the more direct measure, total travel time, can be utilized. In order to reflect the fact that this is an HOV priority treatment, provision will be made for the use of total travel time on a vehicle or passenger basis.

Having identified the criterion function for this optimization model, the next step is to outline certain constraints which will be imposed on the optimization process. The first of these constraints was implied in the preceding paragraph and requires that the total demand on the system be satisfied without diversion. Another constraint is that no system element can be loaded past its maximum capacity. Next, the model should recognize that not all priority vehicles will utilize the reserved section and that some nonpriority vehicles will use the reserved section in violation of its use restrictions. Finally,

the formulation should allow variation in the degree of preferential treatment that is given to priority vehicles.

Basic Model Structure

With the physical system defined, the objective selected, and the solution constraints identified, alternative optimization modeling techniques can now be examined, and a basic model structure formulated. The underlying supply-and-demand nature of this system immediately indicates that some type of commodity assignment technique would be appropriate. Considering the stratification of demand that is necessary to treat passenger travel time, the assignment technique must be capable of recognizing and preserving the distinctions among demands at the various occupancy levels.

This leads into the area of multi-commodity assignment techniques. As a general rule, however, this type of optimization model does not lend itself to the incorporation of the priority variation constraints mentioned in the preceding section. The most notable exception to this rule is the Charnes multi-copy technique [Charnes and Cooper, 1962]. Although this technique does allow the incorporation of special optimization constraints, for this application it is felt that this approach would be somewhat inefficient due to the general nature of the process required for solution of a multi-copy model. Briefly, this technique requires that two separate optimization models be solved for each commodity or copy in the system. For consideration of 5 to 6 levels of passenger occupancy, this would mean solution of 10 to 12 optimization problems.

As a result of the above considerations, examination of an alternative methodology is now in order. The multi-commodity techniques originally considered are in fact special cases of the more general linear programming technique. This linear programming methodology is quite flexible in nature and, as such, has been utilized in a wide variety of applications including modeling of traffic control systems. The primary requirements of this technique are that both the objective function and constraint equations be linear expressions. This requirement does present some problems when modeling traffic systems, but these are relatively minor and can be readily eliminated.

The most significant drawback to this approach is that in the formulation of the objective function, it is necessary to eliminate the distinctions among the various levels of passenger occupancy in determining the optimal system operation. However, this too is a manageable problem. The linear programming model can be utilized as an optimization submodel within a more flexible framework that will account for the multi-commodity nature of the physical system.

The specific problem which arises when attempting to use a linear programming model to optimize (minimize) total travel time on a passenger basis lies in the formulation of the objective function. The general form of this function is given by Equation 3.1 [Sivazlian and Stanfel, 1975, p. 133].

$$\text{Optimize } Z = \sum_{i=1}^n c_i \cdot x_i \quad (3.1)$$

Where: c_i = cost coefficient
 x_i = decision variable.

An expression for the minimization of total travel time on a passenger basis is given by Equation 3.2.

$$\text{Minimize } TTT^P = \sum_{j=1}^m T_j \cdot \left(\sum_{i=1}^n i \cdot x_{ij} \right) \quad (3.2)$$

Where: TTT^P = total passenger travel time
 T_j = unit travel time in section j
 x_{ij} = amount of demand at occupancy level i assigned to section j .

Since the unit travel time (T) is a function of the demand (x), it is not possible to reduce Equation 3.2 to the required form given by Equation 3.1 and still retain the assigned vehicular demand as the decision variable and a scalar cost coefficient. However, this situation may be circumvented by accounting for the passenger occupancy in the unit travel time cost coefficient. This is accomplished by introducing the average passenger occupancy as a scalar multiplier of the unit travel time. The average number of occupants (N) in the vehicles assigned to a given section is

$$N_j = \frac{\sum_{i=1}^n i \cdot x_{ij}}{\sum_{i=1}^n x_{ij}} \quad (3.3)$$

Where: N_j = average number of occupants in all vehicles assigned to section j .

Equation 3.2 can then be rewritten as

$$\text{Minimize } TTP = \sum_{j=1}^m T_j \cdot N_j \cdot \sum_{i=1}^n x_{ij} \quad (3.4)$$

or

$$\text{Minimize } TTP = \sum_{j=1}^m T_j \cdot N_j \cdot x_j \quad (3.5)$$

Where: $x_j = \sum_{i=1}^n x_{ij}$.

Equation 3.5 can then be utilized as the basis for developing an objective function for a linear programming model by considering the cost coefficient (c_j) to be the product of the unit travel time (T_j) and the average occupancy (N_j). The complete development of this objective will be considered in later portions of this chapter.

The preceding demonstration readily identifies the function of the model which will serve as the general framework for the linear programming optimization submodel, that is, to determine the average occupancy of the vehicular flows in both the priority and nonpriority sections. Obviously, since the average occupancies are functions of the assigned flows, and the flow assignments are dependent on the average occupancies, this superstructure model will be an equilibrium assignment technique. That is to say, the model will assume values for the average occupancies of the priority and nonpriority sections,

determine the optimal flow assignments, revise the occupancy estimates, and continue the process until equilibrium has been achieved between the estimated occupancies and the optimal flow assignments.

In summary, the basic structure of the car pool definition model will be a deterministic optimization technique. The overall model will be an equilibrium assignment process with an internal linear programming optimization submodel. The equilibrium model framework will seek a balance between the estimated average occupancies and the assigned optimal flows for the priority and nonpriority sections. The linear programming submodel will determine the optimal flow assignments for the priority and nonpriority sections based on the estimated average occupancies, the system operating characteristics and the related system constraints. The result of the model operation will be a recommendation as to the minimum number of passengers which should be required in order to qualify a vehicle for priority status in conjunction with an HOV priority lane system on an urban freeway.

Development of the Equilibrium Model

Objective

Recalling the statement of the proposed function of the equilibrium model in the previous section, this structure may be viewed as an iterative traffic assignment technique. The purpose of this process is to achieve a balance, or point of equilibrium, between the assigned flows in the priority and nonpriority sections and the average vehicular occupancies for those sections. The iterative nature of this process is necessary since the average section occupancies are functions of the

assigned flows, which are in turn dependent on these average occupancies.

At this point, it would be appropriate to note one deviation from the previous discussion. This is in the definition of the overall objective function for this model. References have been made to total travel time on both a vehicle and a passenger basis. In reality, these measures of effectiveness are not entirely independent. It is convenient to view the total passenger travel time as simply a "weighted" total vehicle travel time. The weighting factor, which must be applied at the stratified flow level, then becomes a method by which the objective function of the model can be defined. If a weighting factor of 1.0 is assumed for all levels of occupancy, the model will use total vehicular travel time as its objective criterion. On the other hand, if this weighting factor is the passenger occupancy at each level, the total passenger travel time will be the figure of merit for the model. These examples by no means exhaust the possible variations of this weighting factor approach. In fact, these factors can be viewed as the relative importance of the flow at each occupancy level, and as such, reflect the opinions of individual users. The two examples that were presented do result in measures with physical meaning, whereas, other formulations would not possess this property. For the remaining discussions pertaining to this model, the selected figure of merit will simply be referred to as the weighted total travel time (TTT^W).

General Structure

The physical system being addressed by this portion of the model is a section of urban freeway with lane(s) reserved for use by

high-occupancy vehicles. This section has certain operating characteristics such as capacity and speed-flow relationships which can be different for the two subsections. The total demand for use of this section has a known distribution of occupancy levels and can be treated as a stratified set of demands. For each of these demand levels, the relative importance is known via the set of flow weighting factors which is specified. This system structure is shown in Figure 3.2

The equilibrium model has a basic iterative structure in which the average subsection weighting factors are balanced with the optimal flow assignments. The technique is initiated with an estimate of the average flow weights for the two subsections. This estimate is then used to determine the optimal flow levels in each subsection through the optimization submodel. These optimal flow levels are in turn used in a demand assignment process, which results in a determination of the actual subsection flow weights. If the estimated and actual flow weights are the same, the process ceases and final recommendations may be formulated. Otherwise, the flow weight estimates are revised, and the process is repeated until equilibrium has been achieved. This overall process is shown as a block diagram in Figure 3.3.

Mathematical Development

As was previously stated, the objective of this portion of the model is to achieve a balance between average subsection flow weights and assigned flows. Assuming that an estimate of the average flow weights for the two subsections is available ($\hat{\theta}_1, \hat{\theta}_2$), the optimal flow

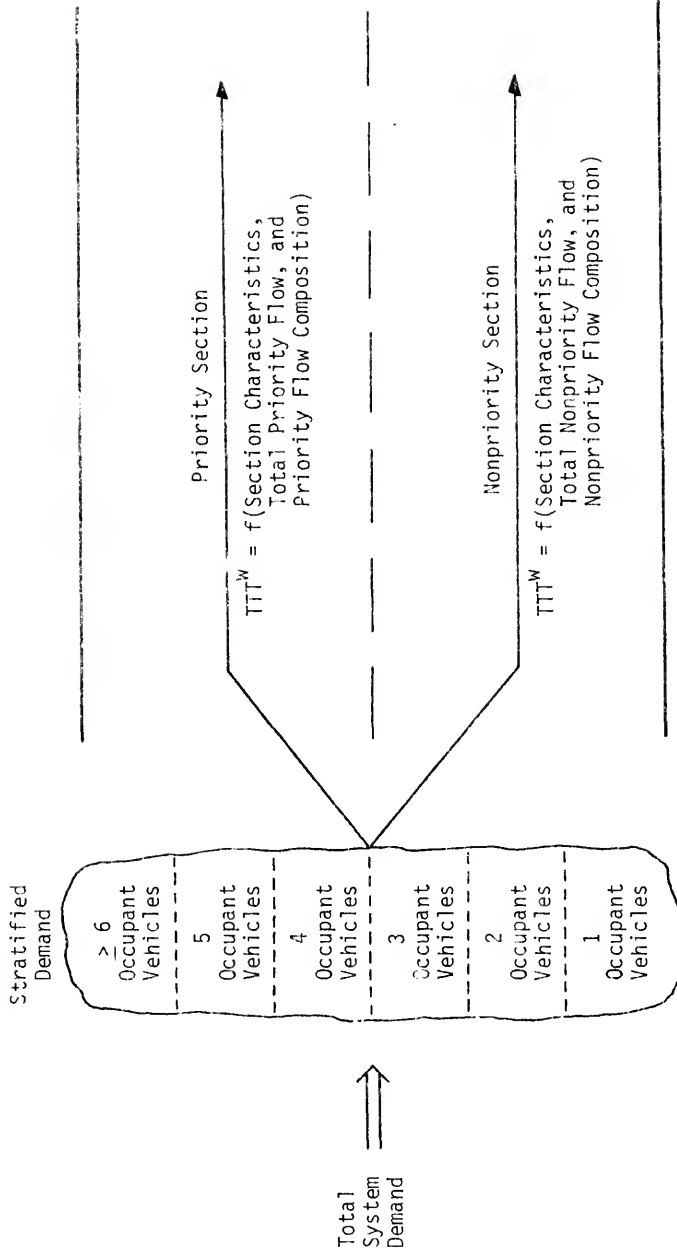


Figure 3.2 CONCEPTUAL SYSTEM FOR THE EQUILIBRIUM MODEL

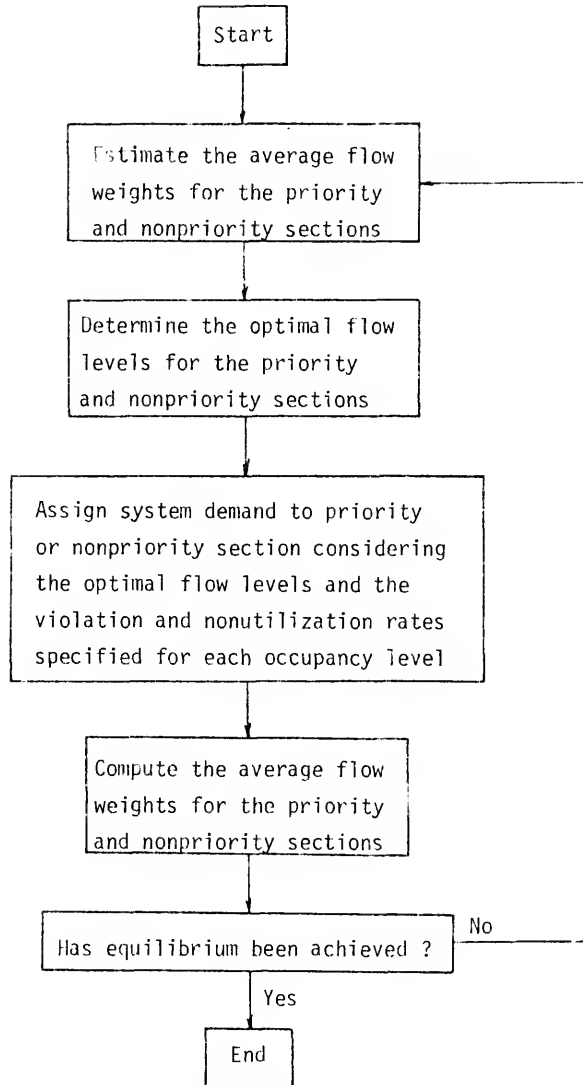


Figure 3.3 STRUCTURE OF THE EQUILIBRIUM MODEL

split between the two sections is known, and the demand assignment has been made, the equilibrium of the model can be tested. The first step in this process is to determine the actual flow weights as indicated in Equation 3.6.

$$O_j = \frac{\sum_{i=1}^n w_i \cdot x_{ij}}{\sum_{i=1}^n x_{ij}} \quad (3.6)$$

Where:

- O_j = actual average flow weight for subsection j
- w_i = flow weight factor for level of occupancy i
- x_{ij} = portion of demand at occupancy level i assigned to the priority section.

Equilibrium is then tested against specified tolerance limits as indicated in Equation 3.7.

$$\frac{|O_j - \hat{O}_j|}{O_j} \times 100\% \leq E \quad (3.7)$$

Where: E = specified maximum error (%) for average flow weight estimates.

If both inequalities hold, equilibrium will have been achieved. If not, the flow weight estimates are revised and the process is repeated. This revision process is indicated in Equation 3.8.

$$\hat{O}_j' = \hat{O}_j + \alpha (O_j - \hat{O}_j) \quad (3.8)$$

Where:

- $\hat{O}_j^i$ = revised estimate of average flow weight in section j
- $\hat{O}_j$ = previous estimate of average flow weight in section j.
- α = specified stepsize for the revision of flow weight estimates.

It should be noted that the revision procedure in Equation 3.8 actually revised the original estimate by some fraction (α) of the difference between the estimated and computed weights as opposed to using the computed values as the new estimates. The rationale here is to reduce the likelihood of developing oscillations which would preclude the achievement of equilibrium. This technique can then be described as a bivariate search technique with a fixed fractional stepsize.

Recalling that the primary motivation for using this equilibrium model structure is to account for the multi-commodity nature of the system, a discussion of how this is achieved would be in order. Since this multi-commodity nature is a result of the desire to incorporate weighting factors for each level of passenger occupancy, reference to Equation 3.6 will show that the flow weight factors are incorporated in the computation of these scalar quantities. It is these scalars which are used as multipliers for the unit travel time cost coefficients in the optimization submodel. Through this process, the stratification of demand is made possible.

The actual determination of the optimal priority and nonpriority subsection flow levels is made by the optimization submodel. At this point, it will be sufficient to say that the equilibrium model supplies this submodel with the physical system parameters, the demand characteristics

and an estimate of the average subsection flow weights. The submodel uses this information to determine the optimal system operation and returns to the equilibrium model information pertaining to the optimal level of flow in each subsection.

With the optimal flow levels supplied by the optimization submodel, the assignment state of the equilibrium process can be carried out. This is basically a direct assignment technique in which flows at the higher levels of occupancy are assigned to the priority section in accordance with the optimal flow levels. This assignment process should, however, account for two observations related to priority treatment operations. These are the violation of the lane use restrictions by nonpriority vehicles and the nonutilization of the reserved lane(s) by qualified priority vehicles.

The violation of the reserved section restrictions by non-qualified vehicles is a phenomenon that has been observed in practically all implementations of HOV priority lanes. Simply stated, this means that some portion of those not qualified as high-occupancy vehicles will use the reserved lane(s) in violation of the use restriction. For the purposes of this model, this can be stated mathematically as in Equations 3.9 and 3.10.

$$x_{i1} \geq \alpha_i d_i \quad (3.9)$$

or conversely,

$$x_{i2} \leq (1 - \alpha_i) \cdot d_i \quad (3.10)$$

Where:

- x_{i1} = amount of flow at occupancy level i which uses the reserved section
- x_{i2} = amount of flow at occupancy level i which uses the nonreserved section
- α_i = proportion of vehicles at occupancy level i which will violate the reserved lane restrictions if that level is not considered high-occupancy
- d_i = total demand at occupancy level i .

Another characteristic which has been observed is that not all qualified vehicles will utilize the reserved section. This factor must also be taken into consideration when assigning system demand. This consideration is, in essence, the complement of the violation rate, as is shown in Equations 3.11 and 3.12.

$$x_{i2} \geq \beta_i \cdot d_i \quad (3.11)$$

or conversely,

$$x_{i1} \leq (1 - \beta_i) \cdot d_i \quad (3.12)$$

Where:

- β_i = proportion of the demand at occupancy level i which will not utilize the reserved sections if that level is considered high-occupancy.

This system is shown pictorially in Figure 3.4.

Summary of the Equilibrium Model

In summary, this model balances the average subsection flow weights and assigned flows to achieve equilibrium. The actual determination of optimal flow levels is the function of the optimization

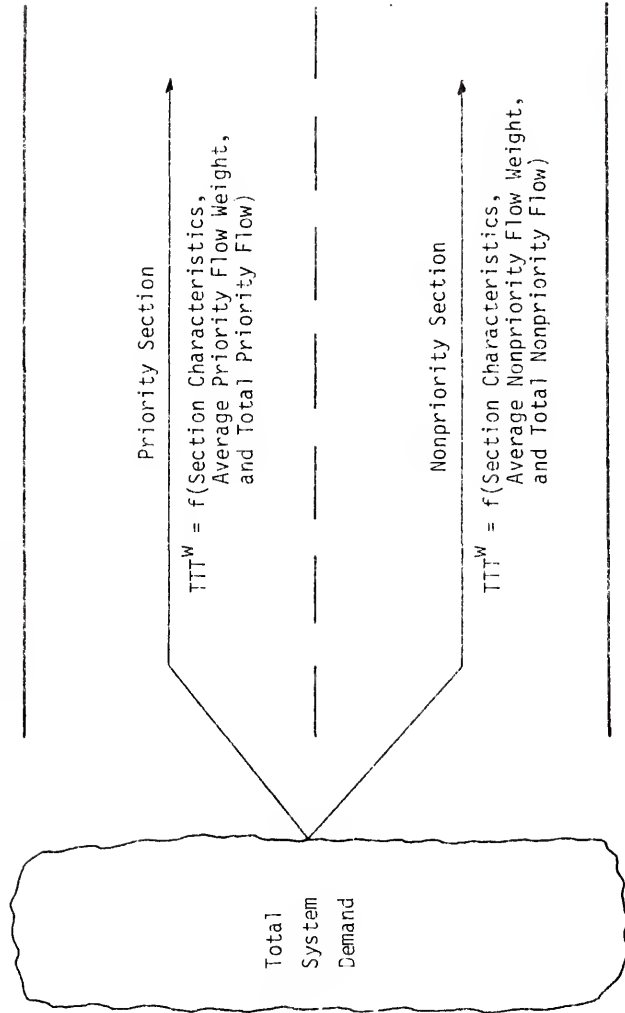


Figure 3.4 CONCEPTUAL SYSTEM FOR THE OPTIMIZATION SUBMODEL

submodel. The flow assignment process takes into account the violation and nonutilization rates for each occupancy level. The output of the total model is then the optimal assignment, by level of occupancy, of the system demand to the priority and nonpriority subsections.

Development of the Optimization Submodel

Objective

As has been previously defined, the objective of the optimization submodel is the determination of the optimal flow levels for the priority and nonpriority subsections. This determination is to be based on the physical system characteristics, total system demand, and the average subsection flow weights as determined by the equilibrium structure. The figure of merit to be utilized in developing the objective function for optimization is the weighted total system travel time (TTT^W).

General Structure

The physical system which is to be considered at this point is similar to the original system. It represents a section of urban freeway which is partitioned into subsections designated for use by priority and nonpriority demand. The section has certain operating characteristics such as capacity and speed-flow relationships which can be different for the subsections. The system demand, however, is not stratified as in the original system. The individual level of occupancy distinction is treated indirectly through the use of the average flow weight estimates.

The modeling approach to this physical system is a basic linear programming technique. In this technique, the objective function (TTT^W) is to be minimized within the limits of certain constraints which describe the physical system limitations and the level of priority which is to be given to the high-occupancy vehicles.

The linear programming process can be thought of as the allocation of a resource or resources in such a manner that some function of this allocation is optimized (maximized or minimized) within the limitations of a set of constraints which are also functions of the allocation [Sivazlian and Stanfel, 1975, p. 133]. As implied by the term "linear" programming, a basic requirement of this technique is that the set of equations describing the objective and constraints be linear expressions. In matrix-vector notation, this structure can be expressed as follows:

$$\text{MAX, MIN } Z = \bar{c} \cdot \bar{x} \quad (3.13)$$

$$\text{Subject to: } \bar{A} \cdot \bar{x} \{ \leq = \geq \} \bar{b} \quad (3.14)$$

$$\bar{x} \geq \bar{0} \quad (3.15)$$

Mathematical Development

With the basic model structure identified, development of a linear programming approach which will meet the stated objectives can proceed. Expressing these required objectives in a format similar to Equations 3.13 to 3.15 the model becomes

$$\text{Minimize } Z = TTT^W$$

- Subject to: (1) Physical system limitations
 (2) Level of priority constraints
 (3) Non-negative flows.

Looking first at the objective function for this model, the weighted total travel time can be expressed initially as follows:

$$TTT^W = \sum_{j=1}^2 T_j \cdot \sum_{i=1}^m w_i \cdot x_{ij} \quad (3.16)$$

Where: T_j = unit travel time in section j
 w_i = flow weight factor for level of occupancy i
 x_{ij} = amount of demand at occupancy level i assigned to section j .

However, as has been shown, this is a multi-commodity structure and can be reduced to a single-commodity format for use in a linear programming approach. This was done by introducing the average flow weights (O_1, O_2) for the priority and nonpriority section flows. With this variable, Equation 3.16 can be reduced to the following.

$$TTT^W = \sum_{j=1}^2 T_j \cdot O_j \cdot x_j \quad (3.17)$$

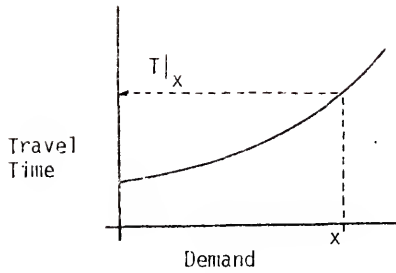
Where: O_j = average flow weight in section j
 x_j = total demand assigned to section j .

This formulation is then parallel in structure to Equation 3.13 considering $\bar{c} = \{T_j, O_j\}$ and $\bar{x} = \{x_j\}$.

A basic relationship of traffic flow that should be considered at this point is that the unit travel time on a roadway is a nonlinear

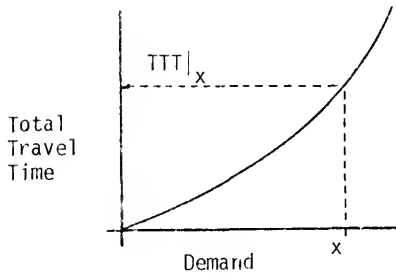
function of the vehicular demand. This means that the objective function in Equation 3.17 is nonlinear since the unit travel time portion (T_j) of the cost coefficient is a function of the decision variable, x_j . This nonlinearity can be removed as follows. First, recognizing that unit travel time (T) is a function of demand (x) as illustrated in Figure 3.5a, the total travel time at any demand x is defined as $TTT|_x = x \cdot T|_x$. Utilizing this as a transformation procedure, the total travel time (TTT) can be expressed as a function of demand. This is illustrated in Figure 3.5b. The weighted total travel time (TTT^W) at any demand x is then equal to $w \cdot TTT|_x$, where w is the flow weight factor. Transforming this TTT function into a TTT^W function, an expression for TTT^W as a function of demand can be developed as shown in Figure 3.5c.

At this point the original nonlinearity in the objective function has been removed, however, the resulting cost coefficients are variable rather than fixed quantities. A relatively simple technique can be utilized to alleviate this problem. This consists of approximating the curve of Figure 3.5c with a series of straight line segments, known as a piecewise linear approximation (PLA) [Wagner, 1975, p. 563]. This approximation process, shown in Figure 3.6, allows the cost function to be treated as a set of linear cost functions. The effective result of this technique is to treat the flow between any two points (i, j) on a roadway as flow on a series of "branches" between these same points as shown in Figure 3.7. Each branch has a maximum capacity, determined by the "break points" in the PLA, and a unit flow cost which is the slope of the PLA segment represented by the particular branch.



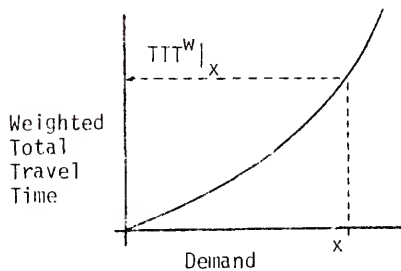
$$TTT|_x = x \cdot T|_x$$

(a) Travel Time vs. Demand



$$TTT^W|_x = w \cdot TTT|_x$$

(b) Total Travel Time vs. Demand



$$TTT^W = f(\text{Demand})$$

(c) Weighted Total Travel Time vs. Demand

Figure 3.5 EVOLUTION OF A WEIGHTED TOTAL TRAVEL TIME VS. DEMAND RELATIONSHIP

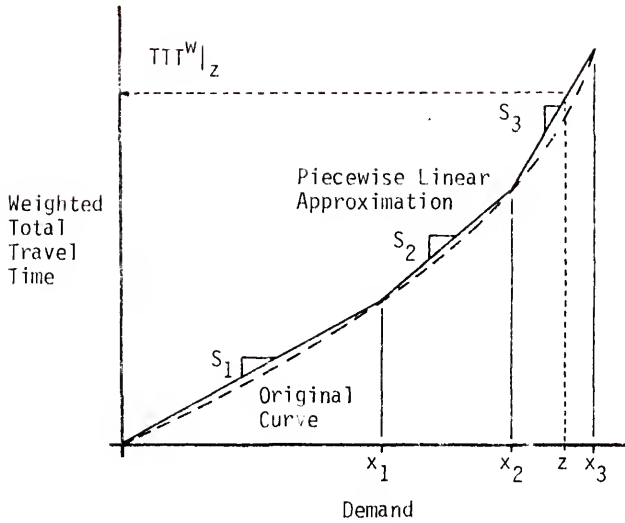


Figure 3.6 PIECEWISE LINEAR APPROXIMATION OF WEIGHTED TOTAL TRAVEL TIME VS. DEMAND

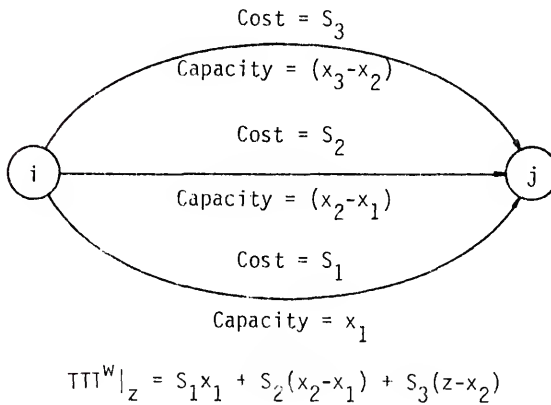


Figure 3.7 EFFECT OF PIECEWISE LINEAR APPROXIMATION

Applying this approximation technique to Equation 3.17, the objective function can be restated as follows:

$$TTT^W = \sum_{j=1}^2 \sum_{k=1}^m S_{jk} \cdot x_{jk} \quad (3.18)$$

Where: S_{jk} = unit flow cost on branch k of section j
 x_{jk} = amount of demand assigned to branch k
of section j

Although the utilization of this technique has transformed the approach to the general class of separable programming, the term linear programming will still be applied to the model. This separable programming technique has merely reduced a nonlinear programming problem to a linear programming format.

In the final objective function formulation, Equation 3.18, the multi-commodity nature of the physical system has been addressed with the use of the average flow weight variable, which is a derivative of the individual occupancy level weights and the assigned section flows. Additionally, the nonlinearities in the original formulation were eliminated through the use of a piecewise linear approximation of a weighted total travel time cost coefficient function. The resulting conceptual system for this formulation is shown in Figure 3.8.

The constraints which are to be imposed on the solution process can now be considered. The general classes of constraints which will be developed are system demand constraints, priority and nonpriority subsection capacity constraints, and the level of priority

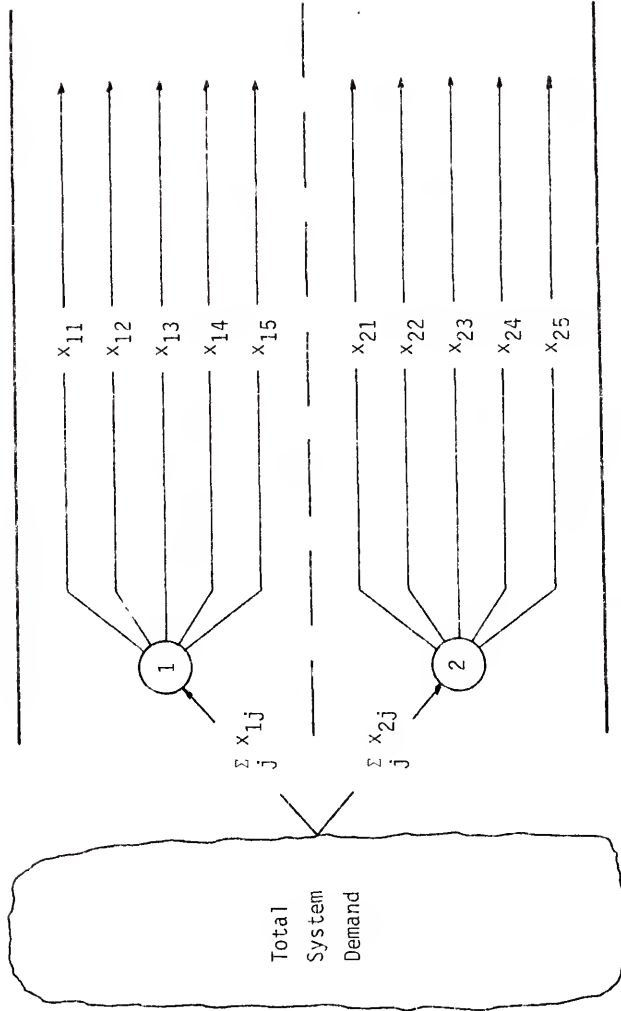


Figure 3.8 CONCEPTUAL STRUCTURE OF THE OPTIMIZATION SUBMODEL

constraints. The speed-flow relationships mentioned in previous sections are omitted from this list since they are reflected in the travel time portion of the cost coefficients for the objective function.

The first type of constraint to be considered is that of satisfaction of the total demand. This constraint is required as a result of a previous decision that no traffic diversion was to be allowed. Thus, the model must accept and consider the total vehicular demand on the system. In terms of the model variables, this constraint is expressed mathematically as follows:

$$\sum_{j=1}^2 \sum_{k=1}^m x_{jk} = \sum_{i=1}^n d_i \quad (3.19)$$

Where: x_{jk} = the amount of demand assigned to branch k of section j
 d_i = the total system demand at occupancy level i.

Next, consideration must be given to the system capacity. Prior to this point, no restriction has been made limiting the system operation in the realm where demand is less than capacity. It is not the intent of this type constraint to impose this restriction on the model. Rather, it is intended to insure that the individual flow branches resulting from the approximation of the weighted total travel time curve are not loaded beyond their individual capacities. This series of constraints can be expressed as follows:

$$x_{jk} \leq c_{jk} \quad \text{for } (j=1,2), (k=1,2,\dots,m) \quad (3.20)$$

Where: x_{jk} = amount of demand assigned to
branch k of section j
 c_{jk} = capacity of branch k of section j.

With these constraints, the total system capacity is indeed restricted, but not to the physical system capacity. It is intended that the upper limit on the assigned subsection flows not be the capacity of the subsection per se, but the maximum level of demand for which the demand speed characteristics can be determined or estimated. This point will be addressed more fully in subsequent portions of this chapter.

The final type of constraint to be considered is that of level of priority. This constraint class is intended to provide a means by which the amount of priority that is given to the high-occupancy vehicles can be controlled. Prior to detailed discussion of these constraints, some general comments are in order.

The first pertains to the manner in which the level of priority is to be measured. Several performance characteristics such as speed, travel, time, and delay are commonly used to evaluate the quality of traffic flow. These characteristics are strongly related to one another, as well as to the ratio of system demand to capacity. This demand to capacity (D/C) ratio is the basic measure of the extent to which a system is loaded. As this ratio increases as a result of increasing demand, the operating speeds tend to decrease and system travel time and delay tend to increase. Recognizing this interdependence and the fact that the basic parameters that have been utilized in the model to this point are the system demands and capacities, selection of the

D/C ratio as the measure of priority level would be appropriate. Specifically, the level of priority will be reflected by the relative D/C ratios for the reserved and nonreserved sections.

Attention must now be directed toward the development of a series of constraints which will provide control over the degree of priority which is to be afforded high-occupancy vehicles. The basic constraint is that the priority section should always operate with a D/C ratio less than that of the nonpriority section. By definition, priority treatment cannot be provided if this constraint is violated. In general terms, this minimum level of priority constraint can be expressed as follows:

$$D/C_n - D/C_p \geq 0 \quad (3.21)$$

Where: D/C_n = demand to capacity ratio for the nonpriority section

D/C_p = demand to capacity ratio for the priority section.

In terms of the model variables, Equation 3.21 becomes

$$\frac{\sum_{k=1}^m x_{2k}}{c_2} - \frac{\sum_{k=1}^m x_{1k}}{c_1} \geq 0 \quad (3.22)$$

Where: x_{jk} = assigned flow on branch k of section j
 c_j = capacity of section j.

However, in some cases it might be desirable to maintain a minimum differential between the D/C ratios for the two sections. In

this event, another constraint which requires a fixed level of priority would be required. This constraint would be of the form

$$D/C_p - D/C_n \geq \alpha \quad (3.23)$$

Where: α = a minimum level of priority differential.

In terms of the model variables, this is expressed as follows:

$$\left[\frac{\sum_{k=1}^m x_{2k}}{c_2} \right] - \left[\frac{\sum_{k=1}^m x_{1k}}{c_1} \right] \geq \alpha \quad (3.24)$$

The third type of priority level constraint addresses the case where it is desired to provide an increasing level of priority as the system becomes more heavily loaded. This condition can be expressed as

$$\beta \cdot (D/C_n) - D/C_p = 0 \quad (0 < \beta \leq 1) \quad (3.25)$$

Where: β = desired ratio of D/C values.

Again, in terms of the model variables,

$$\beta \cdot \left[\frac{\sum_{k=1}^m x_{2k}}{c_2} \right] - \left[\frac{\sum_{k=1}^m x_{1k}}{c_1} \right] \geq 0 \quad (3.26)$$

The final consideration is to allow a decreasing level of priority as the system loading increases. This can be expressed as

$$\frac{1}{1-\rho} \cdot (D/C_n) - D/C_p \geq \frac{\rho}{1-\rho} \quad (0 \leq \rho < 1) \quad (3.27)$$

Where: ρ = offset of the D/C_p vs. D/C_n curve along the D/C_n axis.

Expressing this in terms of the previous model variables,

$$\frac{1}{1-\rho} \cdot \left[\frac{\sum_{k=1}^m x_{2k}}{c_2} \right] - \left[\frac{\sum_{k=1}^m x_{1k}}{c_1} \right] \geq \frac{\rho}{1-\rho} \quad (3.28)$$

In summary, four priority level constraints have been formulated. These are:

Type 0 (Minimum Level of Priority)

$$D/C_n - D/C_p \geq 0 \quad (3.29)$$

Type 1 (Fixed Level of Priority)

$$D/C_n - D/C_p \geq \alpha \quad (3.30)$$

Type 2 (Increasing Level of Priority)

$$\beta \cdot (D/C_n) - D/C_p \geq 0 \quad (3.31)$$

Type 3 (Decreasing Level of Priority)

$$\frac{1}{1-\rho} \cdot (D/C_n) - D/C_p \geq \frac{\rho}{1-\rho} \quad (3.32)$$

These priority level constraints are shown graphically in Figure 3.9.

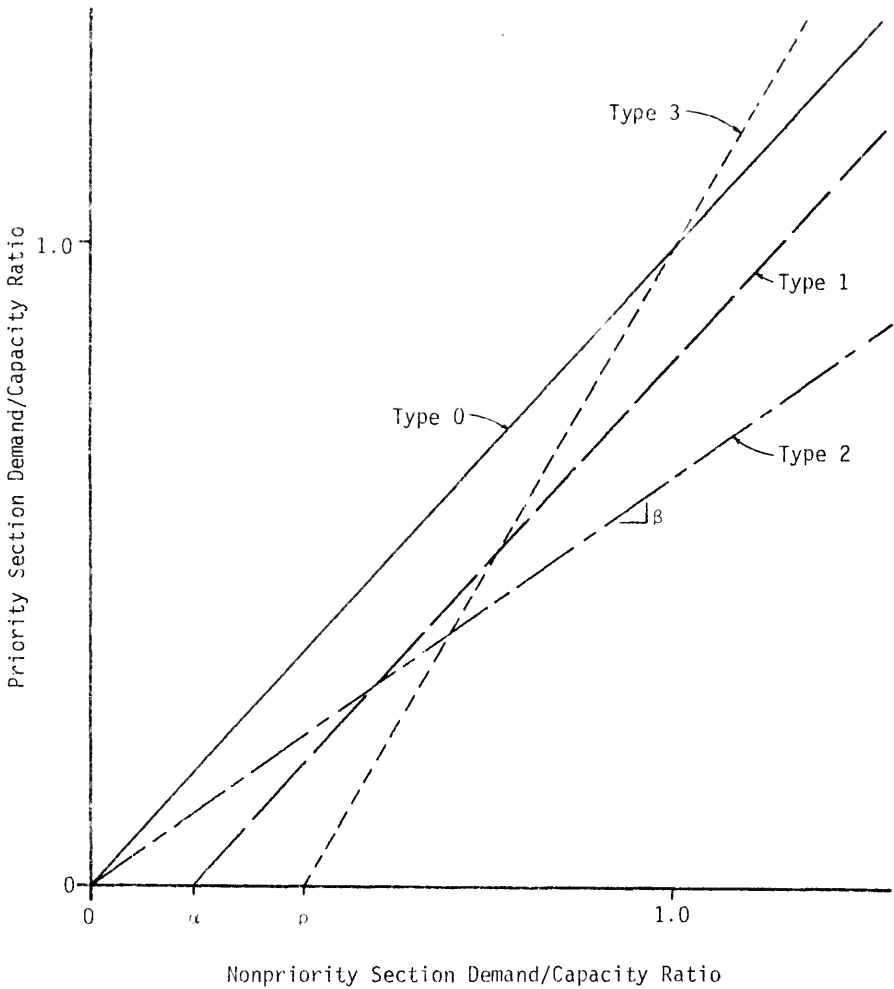


Figure 3.9 LEVEL OF PRIORITY CONSTRAINTS

Naturally, these constraints can be employed independently to define an operating strategy for the priority lane system. A close examination of Figure 3.9 reveals that certain combinations of these constraints also define reasonable operating strategies. These combinations are a constraint Type 0 or 1 in combination with a constraint Type 2 or 3. Also, it may be seen from examination of Equations 3.21 through 3.28 that these relationships can be reduced to an equivalent Type 0 constraint ($\alpha=0$, $\beta=1$, $\rho=0$). Thus, it is apparent that these alternative strategies can be expressed with, at most, two constraint equations. These equations are as follows:

$$\left[\frac{\sum_{k=1}^m x_{2k}}{c_2} \right] - \left[\frac{\sum_{k=1}^m x_{1k}}{c_1} \right] \geq \delta \quad (3.33)$$

$$\lambda \cdot \left[\frac{\sum_{k=1}^m x_{2k}}{c_2} \right] - \left[\frac{\sum_{k=1}^m x_{1k}}{c_1} \right] \geq \tau \quad (3.34)$$

Where: $\delta = 0$ or α

$$(\lambda, \tau) = (\beta, 0) \text{ or } \left(\frac{1}{1-\rho}, \frac{\rho}{1-\rho} \right).$$

This final formulation, Equations 3.33 and 3.34 can then be used as model constraints to control the level of priority which will be given to high-occupancy flow under any operating strategy.

Summary of Optimization Submodel

In summary, the linear programming submodel is a deterministic optimization procedure for identifying the optimal priority and non-priority flows with the objective of minimizing the weighted total travel time. Constraints have been developed for the model which require that all system demand be satisfied, that no travel branch is loaded beyond capacity, and that the level of priority given to the high-occupancy vehicles meets certain criteria. This model structure is shown in matrix form in Table 3.1.

Solution Methodology

Underlying Process

The structure of the model which has been developed for determining an optimal car pool definition for HOV priority lane systems is based on the solution of an equilibrium model with an internal linear programming optimization submodel. For the model as developed, the following steps should be included in the solution technique:

1. Establish system parameters and operating characteristics. This includes determination of the capacities and speed-flow relationships for the priority and nonpriority sections, as well as ascertaining the vehicular demand by level of occupancy.
2. Develop optimization constraints. These are both the demand and capacity constraints, and the operating strategy constraints.

Table 3.1 OPTIMIZATION SUBMODEL STRUCTURE

Decision Variable										Sense	RHS
x_{11}	x_{12}	x_{13}	x_{14}	x_{15}	x_{21}	x_{22}	x_{23}	x_{24}	x_{25}		
$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\geq$	0
$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\frac{1}{c_n}$	$\geq$	α
$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$\frac{\beta}{c_n}$	$\frac{\beta}{c_n}$	$\frac{\beta}{c_n}$	$\frac{\beta}{c_n}$	$\frac{\beta}{c_n}$	$\geq$	0
$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$-\frac{1}{c_p}$	$\frac{1}{1-\rho} \frac{1}{c_n}$	$\frac{1}{1-\rho} \frac{1}{c_n}$	$\frac{1}{1-\rho} \frac{1}{c_n}$	$\frac{1}{1-\rho} \frac{1}{c_n}$	$\frac{1}{1-\rho} \frac{1}{c_n}$	$\geq$	$\frac{\rho}{1-\rho}$
1	1	1	1	1	1	1	1	1	1	=	xd_i
1										$\leq$	c_{11}
	1									$\leq$	c_{12}
		1								$\leq$	c_{13}
			1							$\leq$	c_{14}
				1						$\leq$	c_{15}
					1					$\leq$	c_{21}
						1				$\leq$	c_{22}
							1			$\leq$	c_{23}
								1		$\leq$	c_{24}
									1	$\leq$	c_{25}
s_{11}	s_{12}	s_{13}	s_{14}	s_{15}	s_{21}	s_{22}	s_{23}	s_{24}	s_{25}	=	Z

3. Estimate average section flow weights. These estimates will be refined within the equilibrium model operation.
4. Develop cost coefficients for objective function of optimization submodel. These are based on the speed-flow relationships and flow weight estimates.
5. Find optimal flow levels for priority and non-priority sections. This requires solution of the optimization submodel.
6. Determine actual section flow weights. This requires assignment of the demand to the sections considering the optimal flow levels and the violation and nonutilization factors.
7. Check for equilibrium. If the original flow weight estimates compare favorably with the actual values, the process can be terminated. If not, revise the flow weight estimates and the objective function cost coefficients and repeat steps 5 through 7 until equilibrium has been achieved.
8. Formulate recommendations. In this final step, the results of the optimization process are examined to develop a recommended car pool definition.

This solution process is illustrated in Figure 3.10.

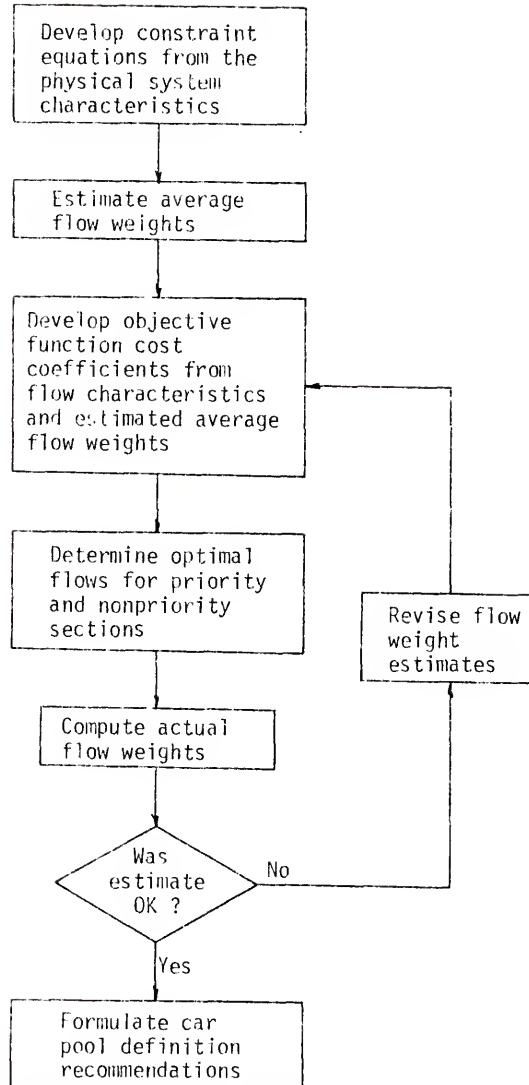


Figure 3.10 CAR POOL DEFINITION MODEL SOLUTION PROCESS

Recommended Techniques

Specific techniques which can be utilized in this solution process have been left to the discretion of model users since they will vary depending on the particular application. However, the following considerations should serve as a set of general guidelines.

In the previous discussion, reference has been made to speed-flow characteristics of the system. For operating at levels below the point where the system demand is equal to the capacity ($D/C \leq 1.0$), this relationship can be determined by direct field measurements, or a general relationship can be obtained from any one of several reference sources. In most cases, however, HOV priority techniques are not considered until such time as the demand approaches or exceeds the capacity. Additionally, for a particular application no assurances can be given that optimal system performance will allow both the priority and nonpriority sections to operate with demand less than capacity. For these reasons, the model itself does not require operation with D/C ratios less than 1.0.

In order to permit the model to treat the oversaturated condition, the demand-speed relationships for this range must be provided. This is by no means a simple task since system demand is difficult to measure in most cases. It is suggested that consideration be given to the use of current travel time prediction techniques to extend the speed-flow relationships into the oversaturated region, where they become speed-demand relationships. Detailed considerations for this extension are presented in Appendix A of this report. At this point,

it will be sufficient to say that the operational characteristics in the oversaturated range can be determined by interfacing the speed or travel time relationships for the two operating regions.

A second point to consider with respect to the speed-demand relationship pertains to the use of the piecewise linear approximation (PLA) to the weighted total travel time vs. demand curve as developed earlier. The use of this technique results in the assumption of constant speed operation within the range of each PLA segment or branch. The effect of this can be minimized by the number of PLA branches involved and the proper selection of the intersection points for the line segments. Traditionally, a 3-segment PLA has been utilized to describe the undersaturated portion of this relationship. Considering the extension into the oversaturated region, it is suggested that a 4- or possibly 5-segment PLA would be appropriate for the purposes of this model. Additionally, selection of the line segments in such a manner as to concentrate relatively short branches about the expected operating range will improve the accuracy of the analysis process.

With regard to the equilibrium model, two points should be considered with respect to the average flow weights. First, for an initial estimate it is suggested that the overall average flow weight be used for both sections. This will key the initial estimate to the occupancy level distribution and, if all flow weights are equal to 1.0, as would be the case for minimum vehicle travel time, only one iteration of the equilibrium model will be necessary. Second, in revising the flow weight estimates based on subsequent computed values, it is recommended that the stepsize used in Equation 3.8 be in

the range of 0.65 to 0.85. The experience gained during the various applications of the proposed model indicated that lower values tended to retard the equilibrium process, while higher values seemed to increase the likelihood of developing an oscillatory pattern which can prevent proper convergence of the model.

The solution of the linear programming submodel can be accomplished with a variety of techniques. These techniques are well documented in the literature, and several procedures are available for computer solution at most major installations (Shamblin and Stevens, 1974, p. 295]. Individual users should consider utilization of those techniques with which they are most familiar.

This final set of comments deals with the development of a recommended car pool definition. The information available at the end of the equilibrium model process is an optimal assignment of flow, by level of occupancy, to the reserved and nonreserved sections. As it is unlikely that these optimal flows will break on an occupancy level boundary, the boundaries on either side of the optimal configuration must be investigated. When this is done, the boundary with the lesser deviation from the optimal condition should be recommended as the car pool definition provided that the priority level constraints would not be violated. Under some conditions, the less restrictive definition (lower boundary) can result in the D/C ratio actually being higher for the priority section than for the nonpriority section. If this is the case, a check of the system operation at the less restrictive definition with user optimization (equal demand/capacity ratios) should be made and compared with the more restrictive definition as before.

Program CARPOOL

One final note is that the procedures and techniques previously presented have been used to develop algorithms for application in a computerized car pool definition model, CARPOOL. This computerized model will be utilized for the example applications to be presented in subsequent chapters of this report.

No documentation of the program operation will be presented here, inasmuch as its algorithmic structure is based directly on previous considerations. However, it should be noted that the program was written in the FORTRAN IV programming language and was developed for operation on an IBM System 370/165. A block diagram of the program operation is presented in Figure 3.11 for additional reader information.

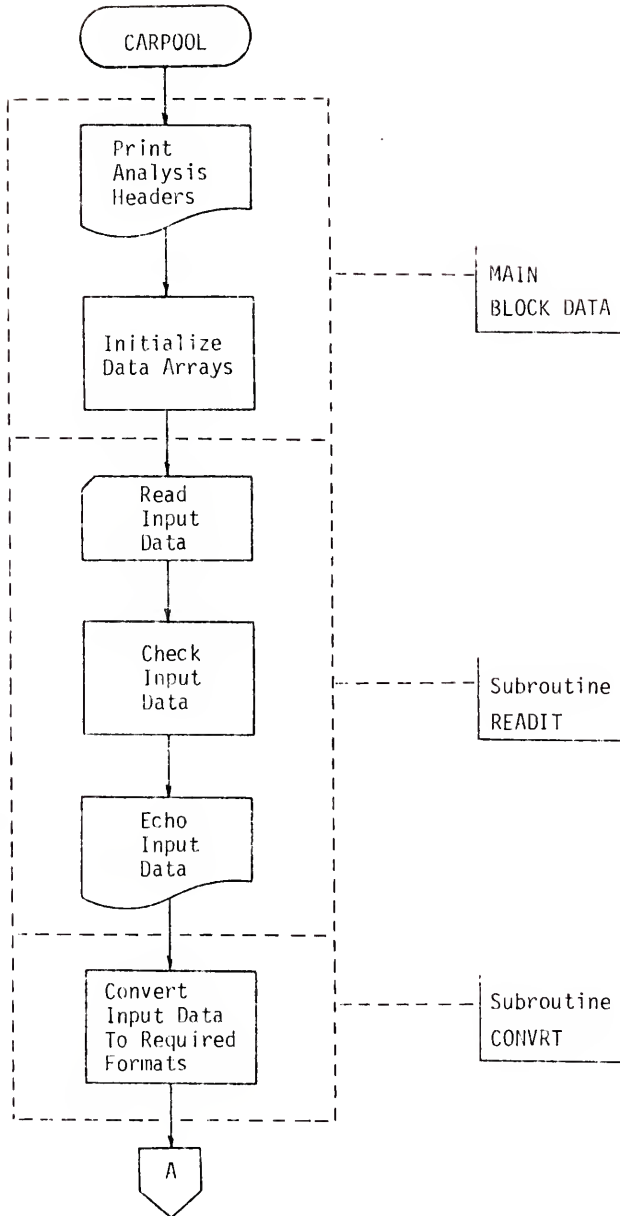


Figure 3.11 BLOCK DIAGRAM FOR PROGRAM CARPOOL

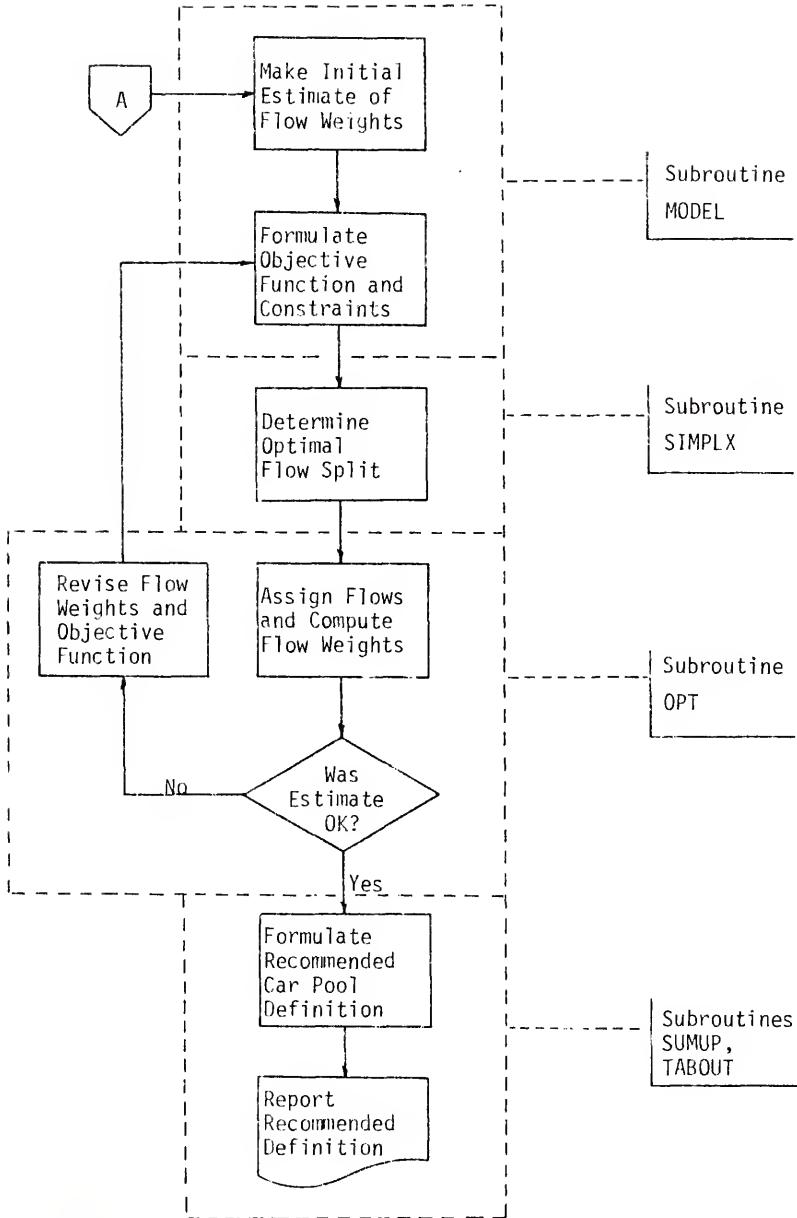


Figure 3.11 - continued

CHAPTER 4

VALIDATION AND APPLICATION OF THE CAR POOL DEFINITION MODEL

Introduction

Objectives

A natural concern associated with the use of any modeling technique, either mathematical or physical, is how well the model reflects the operation of the full-scale system. If the model is inaccurate or does not consider all system aspects, the results of any investigation with the model will be, at best, highly questionable. An additional concern is whether the scope of the model is sufficient for application to the particular problem under investigation. As a general rule, it is more desirable to develop a model in such a manner that application to a variety of situations or physical variations is possible, rather than to limit the application of the model to a pre-determined case.

This chapter addresses these concerns with respect to the car pool definition model which has been proposed. A demonstration of how well the model represents the physical system operation will be presented in the form of a validation process. The scope of the model will be defined through a discussion of the potential applications of the proposed technique. Additionally, some general guidelines for consideration with respect to priority lane operations will be presented.

Organization

The first area to be addressed is the validity of the proposed model. This section will present a number of considerations in support of the overall accuracy of the modeling process. Next, the flexibility of the model will be demonstrated. This demonstration will consist of an examination of the basic application of the model and identification of other considerations which may be investigated with the model. Finally, the general guidelines will be presented.

Validation of the Model

Validation Methodology

Simply stated, validation of a mathematical model consists of verifying that known physical system operations are adequately reflected by the model. The validation process, in and of itself, cannot make a strong positive statement with respect to the accuracy of the modeling technique. The fact that a model can reproduce known conditions does not insure that it can reliably predict operations under other conditions. However, an absence of the ability to reproduce these known conditions does make an extremely strong negative statement about the validity of a model.

Traditionally, the validation process has consisted of applying the model to an existing system for which the operating characteristics are known, and determining how well the model reproduces this system operation. However, a somewhat different approach to the validation of the car pool definition is required. This is necessitated by the fact that the required data for an existing system are not available,

and the field implementation and evaluations necessary to produce them are beyond the scope of this effort. The validation of this model will then consist of a series of sensitivity analyses, designed to demonstrate the effects of varying system parameters on the predicted system operation, and a comparison of the results obtained with the proposed technique and currently accepted simulation models, which will show that the model accurately reflects the system operation.

Sensitivity Analyses

In this section, the sensitivity of the car pool definition model to varying system operating parameters will be addressed. Through this investigation, the degree to which the model predictions conform to accepted traffic flow relationships will be demonstrated. These sensitivity analyses will be limited to those parameters which are scalar quantities or to relationships which may be described by a scalar quantity. Specifically, this section will present the sensitivity of the model to variations in (1) the priority section capacity, (2) the nonpriority section capacity, (3) the total system demand, and (4) the demand distribution with respect to the level of occupancy.

Prior to beginning the sensitivity analyses, it is necessary to establish a base condition or basic system for subsequent variation. For this purpose, a portion of an HOV priority land system currently in operation on I-95 in Miami, Florida, was selected. This test system is 16,910 feet (3.07 miles) in length and has 1 reserved lane and 4 non-reserved lanes. The estimated capacity of this system is 1,500 vehicles per hour (vph) in the reserved section and 8,000 vehicles per hour in

the nonreserved section. The free flow operating speed for each section was assumed to be 60 miles per hour (mph). The total peak hour demand on this system is 8,463 vehicles, and the violation and nonutilization rates were assumed to be 10% and 15%, respectively, for each occupancy level. This test section is shown in Figure 4.1, and the operating characteristics and demand description are presented in Tables 4.1 and 4.2. The speed-demand relationship developed in Appendix A was adapted for this system and is presented in Figure 4.2.

An initial evaluation of this system indicates that for minimum passenger travel time, a car pool definition of 3 persons per vehicle (ppv) is preferable, with the total travel time equal to 914 passenger hours. For minimum vehicle hours of travel, a car pool definition of 3 or 2 ppv results in 684 vehicle hours of travel, if user optimization is assumed for the lower definition. At optimality, the minimum total passenger and vehicular travel times were found to be 913 and 684 hours, respectively.

A widely accepted traffic flow relationship is that the total travel time on a roadway has an inverse curvilinear relationship with the roadway capacity when other factors remain constant [Highway Research Board (HRB), 1965]. It can be inferred from this that if the capacity of the priority section of the basic system were decreased, the optimal total travel time would be increased.

In order to demonstrate that this relationship is reflected in the car pool definition model predictions, a series of evaluations were performed at various priority capacity levels, ranging from 70% to 130%

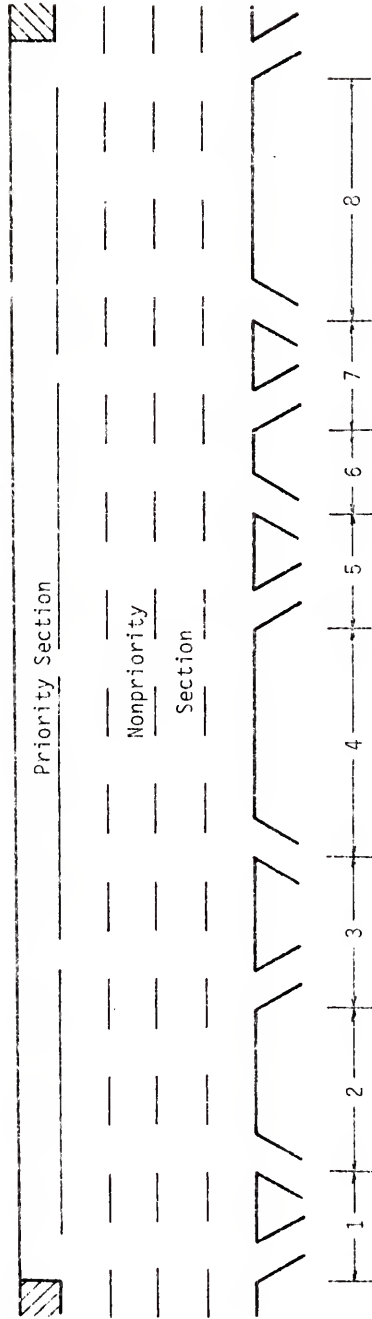


Figure 4.1 SCHEMATIC OF THE TEST SECTION FOR THE
CAR POOL MODEL SENSITIVITY ANALYSES

Table 4.1 SUMMARY OF THE TEST SYSTEM OPERATING CHARACTERISTICS
FOR THE CAR POOL MODEL SENSITIVITY ANALYSES

Section	Length (feet)	Number of Lanes		Capacity (vph)		Free Flow Speed (mph)	
		Reserved	Non- reserved	Reserved	Non- reserved	Reserved	Non- reserved
1	448	1	4	1500	8000	60	60
2	2577	1	4	1500	8000	60	60
3	2075	1	4	1500	8000	60	60
4	3091	1	4	1500	8000	60	60
5	1644	1	4	1500	8000	60	60
6	1054	1	4	1500	8000	60	60
7	1506	1	4	1500	8000	60	60
8	3795	1	4	1500	8000	60	60
Total	16,190	NA	NA	NA	NA	NA	NA

Table 4.2 SUMMARY OF THE DEMAND CHARACTERISTICS FOR
THE CAR POOL MODEL SENSITIVITY ANALYSES

Level of Occupancy (ppv)	Demand		Violation Rate (%)	Nonutiliza- tion Rate (%)
	Vehicles	%		
1	6287	74.29	10.00	15.00
2	1613	19.06	10.00	15.00
3	448	5.29	10.00	15.00
4	76	0.90	10.00	15.00
5	39	0.46	10.00	15.00
6	0	0.00	10.00	15.00
Total	8463	100.00	NA	NA

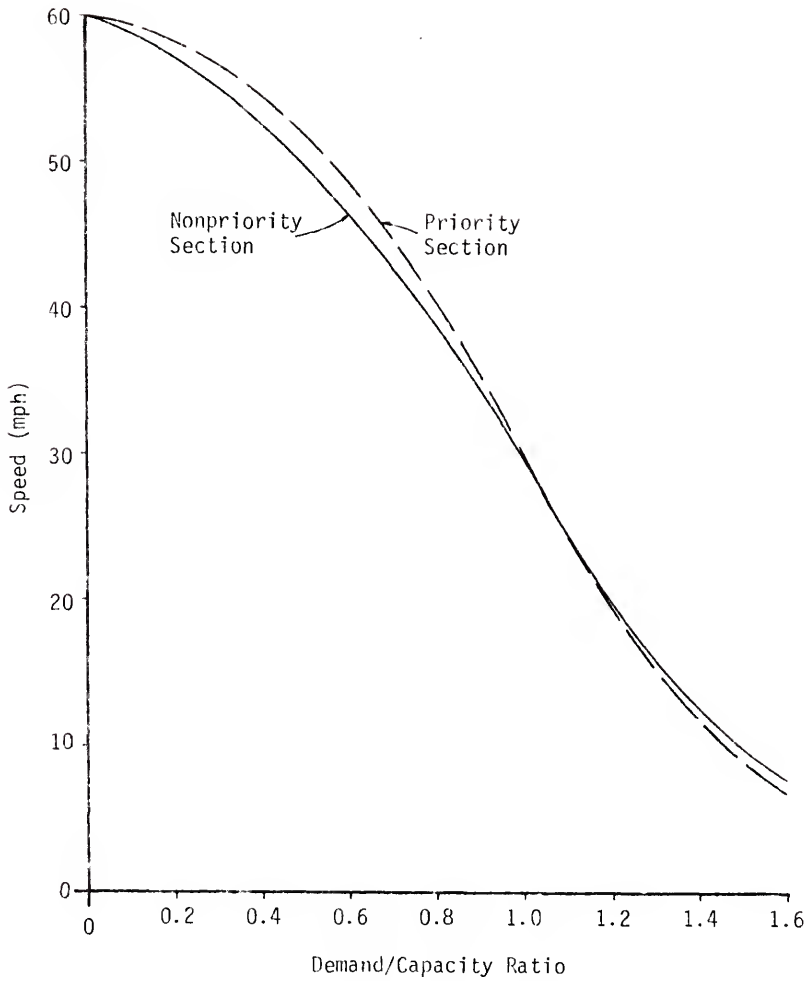


Figure 4.2 ASSUMED DEMAND-SPEED RELATIONSHIPS FOR THE CAR POOL MODEL SENSITIVITY ANALYSES

of the basic system value. The results of these analyses, presented in Table 4.3, show that this relationship is maintained within the mathematical model. The expected curvilinear nature of this relationship and the sensitivity of the model to priority capacity variations are both shown in Figure 4.3.

This same relationship of total travel time and capacity should also hold true for variations in the capacity of the nonpriority section. To ascertain that this is indeed the case, a similar series of analyses were conducted for varying levels of nonpriority capacity, again ranging from 70% to 130% of the basic system value. The results of these analyses are presented in Table 4.4, and again reflect the expected relationship. The sensitivity of the model to nonpriority capacity variations is shown in Figure 4.4.

A second relationship that should be reflected in a valid traffic flow model is the effect of demand on total travel time. As the vehicular demand on a section of roadway increases, the total travel time also increases, but in a nonlinear fashion. In order to demonstrate the validity of the model with respect to this relationship, a third series of analyses were conducted in which the total demand was varied within the range of the basic system demand $\pm 30\%$, and all other factors were held constant. The results of these analyses, contained in Table 4.5, again demonstrate that this basic relationship is reflected by the mathematical model. The nonlinear nature of this relationship and the sensitivity of the model can be seen in Figure 4.5.

A final consideration in this series of sensitivity analyses is the effect of shifts in the distribution of the total demand with

Table 4.3 OPTIMAL SYSTEM OPERATION FOR VARYING
PRIORITY SECTION CAPACITIES

Capacity (vph)	Total Vehicle Hours			Total Passenger Hours		
	Reserved	Non-reserved	Total	Reserved	Non-reserved	Total
1050	73.5	710.7	784.2	120.4	927.3	1047.7
1200	84.1	658.9	743.0	151.9	839.3	991.2
1350	94.6	607.7	701.7	183.4	753.8	937.2
1500	103.6	580.5	684.1	189.7	724.0	913.7
1650	111.5	568.1	679.6	182.0	724.0	906.0
1800	119.1	556.1	675.2	174.4	724.0	898.4
1950	126.3	544.4	670.7	166.7	724.0	890.7

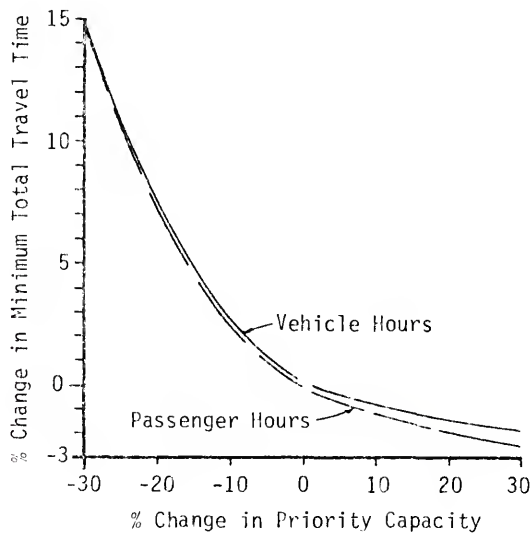


Figure 4.3 EFFECT OF VARYING PRIORITY SECTION CAPACITIES
ON MINIMUM TOTAL SYSTEM TRAVEL TIME

Table 4.4 OPTIMAL SYSTEM OPERATION FOR VARYING
NONPRIORITY SECTION CAPACITIES

Capacity (vph)	Total Vehicle Hours			Total Passenger Hours		
	Reserved	Non-reserved	Total	Reserved	Non-reserved	Total
5600	273.9	1056.7	1330.6	544.7	1240.5	1785.2
6400	105.1	990.1	1095.2	208.6	1207.6	1416.2
7200	105.1	772.7	877.8	208.6	942.4	1151.0
8000	103.6	580.5	684.1	189.7	724.0	913.7
8800	92.5	570.4	662.9	87.6	801.5	889.1
9600	83.1	558.6	641.7	87.6	772.8	860.4
10400	74.9	545.4	620.3	87.6	744.1	831.7

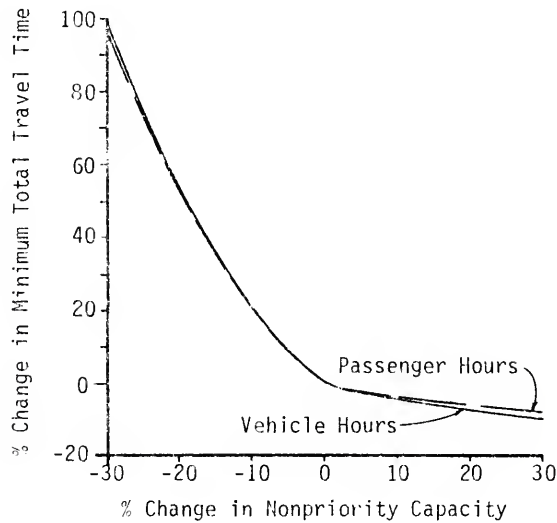


Figure 4.4 EFFECT OF VARYING NONPRIORITY SECTION CAPACITIES
ON MINIMUM TOTAL SYSTEM TRAVEL TIME

Table 4.5 OPTIMAL SYSTEM OPERATION FOR VARYING
LEVELS OF VEHICULAR DEMAND

Demand (veh)	Total Vehicle Hours			Total Passenger Hours		
	Reserved	Non-reserved	Total	Reserved	Non-reserved	Total
5924	60.9	341.1	402.0	103.8	411.8	515.6
6770	75.1	420.9	496.0	106.0	555.4	661.4
7617	89.4	500.7	590.1	97.9	690.9	788.8
8463	103.6	580.5	684.1	189.7	724.0	913.7
9309	105.1	879.9	985.0	205.0	1089.8	1294.8
10156	105.1	1204.9	1310.0	194.9	1521.9	1716.8
11002	127.4	1509.6	1637.0	230.1	1923.3	2153.4

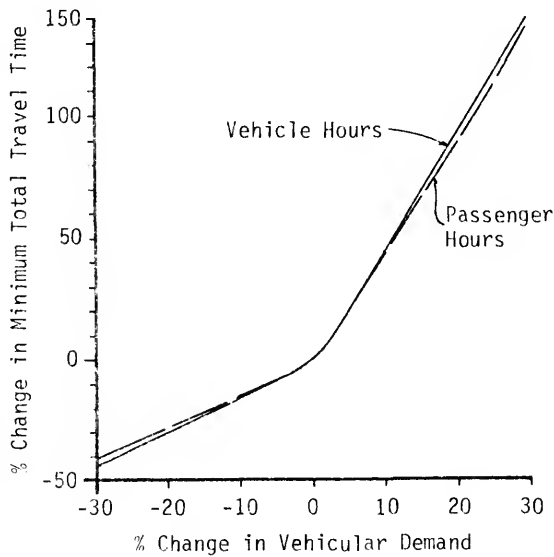


Figure 4.5 EFFECT OF VARYING VEHICULAR DEMANDS
ON MINIMUM TOTAL SYSTEM TRAVEL TIME

respect to the level of occupancy. As the proportion of single-occupant vehicles in the traffic stream increases, the relative passenger demand on the system decreases for constant total vehicular demand. This shift in distribution should have no effect on the minimum vehicular travel time, since the total vehicular demand is fixed. However, since the person-demand is decreasing, the optimal total passenger travel time should also be decreasing. The demand distribution originally assumed for the basic system, Table 4.2, cannot be directly expressed as a scalar quantity for this analysis. However, a plot of these data seems to indicate that they are distributed with an approximate negative-exponential relationship, which may be described with a scalar parameter. A mathematical expression of this type can then be used to approximate this demand distribution. The general form of the expression is given by Equation 4.1.

$$(\% \geq n) = 100e^{-\rho(n-1)} \quad (4.1)$$

Where: $(\% \geq n)$ = percent of the demand with n or more occupants

ρ = constant determining the relative curvature of the relationship.

This shifted negative-exponential relationship approximates the original demand distribution at a value of $\rho \approx 1.30$.

With this expression for the demand distribution, additional distributions can be developed by varying the parameter ρ , which is a scalar quantity. This procedure will permit a sensitivity analysis to be performed for the effect of shifts in the demand distribution. For the purposes of this sensitivity analysis, values of ρ were selected

over the range of 1.1 to 1.6. This distribution will show a shift into single-occupant vehicles for increasing values of ρ . The original cumulative demand distribution and the variation introduced with this expression are shown in Figure 4.6.

Applying these occupancy distributions to a fixed total demand, the effect of occupancy shifts can be examined. As is shown in Table 4.6, the hypothesized relationship between demand distribution and total travel time is reflected by the optimization process. The sensitivity of the model to shifts in the occupancy distribution can be seen in Figure 4.7.

Comparison with Simulation Technique

Another technique which can be used to infer the validity of the car pool definition model is a comparison with a currently accepted simulation technique. For this purpose, the PRIFRE priority lane simulation model [Minister et al., 1973] was selected to provide additional verification of the proposed optimization procedure. If both models are applied to a system in such a manner as to avoid conflict between the basic assumptions of the models, the resulting predictions of system operation should be comparable.

In order to compare these models on a common basis and eliminate discrepancies which might result from conflicting assumptions of the two techniques, the test case must be carefully defined. The system which was developed for this comparison is a priority lane treatment 2 miles in length with 1 reserved lane and 2 nonreserved lanes. The capacities of the reserved and nonreserved sections were assumed to be

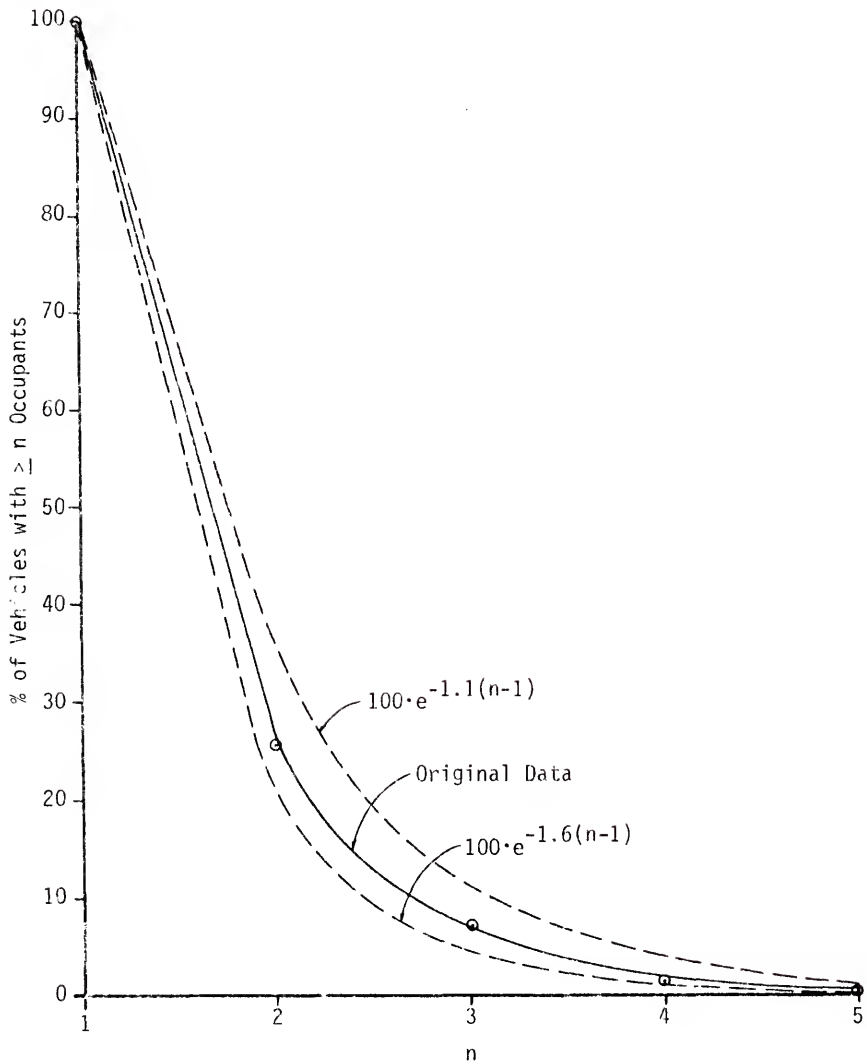
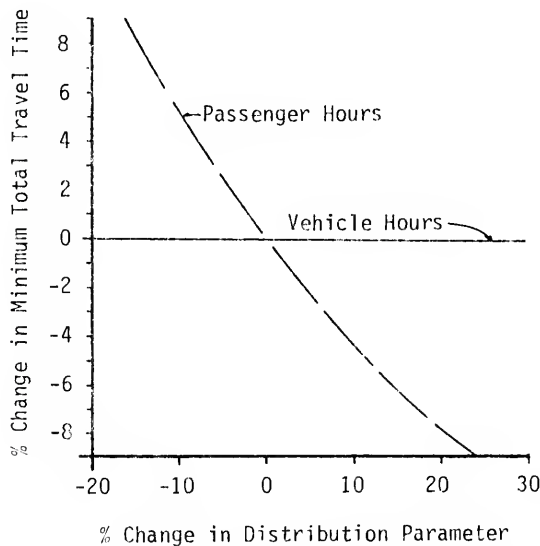


Figure 4.6 DISTRIBUTION OF VEHICULAR DEMAND
BY LEVEL OF PASSENGER OCCUPANCY

Table 4.6 OPTIMAL SYSTEM OPERATION FOR
VARYING DEMAND DISTRIBUTIONS

ρ	Total Vehicle Hours			Total Passenger Hours		
	Reserved	Non-Reserved	Total	Reserved	Non-Reserved	Total
1.1	103.6	580.5	684.1	214.3	801.9	1016.2
1.2	103.6	580.5	684.1	203.5	768.4	971.9
1.3	103.6	580.5	684.1	195.3	739.2	934.5
1.4	103.6	580.5	684.1	186.6	716.5	903.1
1.5	103.6	580.5	684.1	176.4	700.3	876.7
1.6	103.6	580.5	684.1	168.0	685.6	853.6

$$(\% \geq n) = 100 \cdot e^{-\rho} (n-1)$$

Figure 4.7 EFFECT OF VARYING DEMAND DISTRIBUTIONS ON
MINIMUM TOTAL SYSTEM TRAVEL TIME

1,600 vph and 3,700 vph, respectively, and the speed-demand relationships used in the previous section were adopted for this comparison. The total demand was assumed to be constant at 4,500 vph, and the occupancy distribution was allowed to vary over the range identified in the last section of the sensitivity analyses. The resulting vehicle and passenger demands are shown in Table 4.7. Finally, for this comparison, a car pool definition of 2 ppv was assumed for the simulation model, and minimum vehicular travel time was used as the objective criterion in the optimization model.

Both models were applied to this test system for each of the 5 occupancy distributions. As can be seen in Table 4.8 and Figure 4.8, the results of these analyses show that the predicted system operations were similar for both models. Comparison of these results indicates that the average discrepancy was 8.4 vehicle-hours or 2.7%. The maximum difference of 17 vehicle-hours (5.3%) was observed at the apparent discontinuity in the predictions of the PRIFRE model and is possibly attributable to that model's treatment of the piecewise linear approximation of the demand-speed curve.

Applications of the Model

Objective

In this section, a number of potential areas of application for the car pool definition model will be examined. This presentation will not be a series of "cookbook" procedures; rather, it will serve to identify various ways in which the model can contribute to investigations of priority lane treatments. Although it is quite possible that additional applications will arise in the considerations of a

Table 4.7 TOTAL VEHICULAR AND PASSENGER DEMANDS
FOR COMPARISON OF CARPOOL AND PRIFRE

p	Flow Units	Level of Occupancy					Total
		1	2	3	4	5	
1.1	Vehicles	3002	999	333	111	55	4500
	Persons	3002	1998	999	444	275	6718
1.2	Vehicles	3145	947	285	85	37	4500
	Persons	3145	1894	855	344	185	6423
1.3	Vehicles	3274	892	243	66	25	4500
	Persons	3274	1784	729	264	125	6176
1.4	Vehicles	3390	836	206	51	17	4500
	Persons	3390	1672	618	204	85	5969
1.5	Vehicles	3496	780	174	39	11	4500
	Persons	3496	1560	522	156	55	5789

$$(\% > n) = 100 \cdot e^{-p(n-1)}$$

Table 4.3 TOTAL TRAVEL TIMES PREDICTED BY
CARPOOL AND PRIFRE MODELS

ρ	Total Vehicle Hours		% Deviation
	CARPOOL	PRIFRE	
1.1	310	307	0.98
1.2	297	295	0.68
1.3	303	320	-5.31
1.4	323	307	5.21
1.5	348	344	1.16

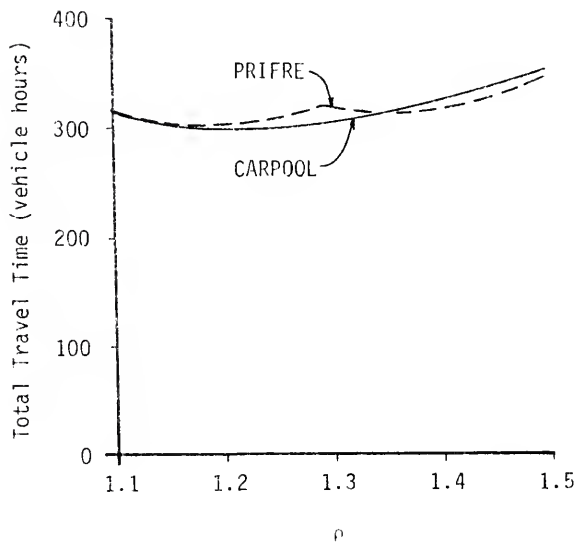


Figure 4.8 COMPARISON OF TOTAL TRAVEL TIME PREDICTIONS
OF CARPOOL AND PRIFRE MODELS

particular system, the following discussion will address those areas which are of more general interest.

Basic Car Pool Definition

The simplest and most obvious application of this model is the development of a basic car pool definition; that is, a single definition to be applied throughout the system for the full period of operation. It is this fixed definition concept that has been adopted for all HOV priority lane systems implemented to date.

An application of the model for this purpose is reasonably straightforward, as has been discussed. One primary area of concern, however, should be the development of the system demand-speed relationships for oversaturated operation. If it is likely that the freeway system will operate with demands near or in excess of the capacity, as would be true in many cases, estimation of these operating characteristics is necessary to allow the total system demand to be considered in the optimization process. Additionally, it might be beneficial or necessary in some instances to allow congestion in some portion of the system in order to achieve overall optimality. A convenient method of extending the basic speed-flow relationships into the oversaturated region is the utilization of a travel time estimation procedure. These procedures were developed primarily for use in the transportation planning process; however, they are readily adaptable for use in this model, as is discussed in Appendix A.

Another area in which caution would be advised is the segmentation of the freeway system into homogeneous subsystems. If the system

is reasonably stable with respect to level of demand and capacity throughout its length, this subdivision process would not be necessary. On the other hand, if these parameters vary to a significant degree, i.e., changes in the number of lanes or substantial changes in the demand, it would be advisable to conduct the system analysis as a series of subsystem analyses. This multiple analysis process would avoid the situation in which the demand or capacity within any section would be incorrectly considered.

Spatial Variation of the Car Pool Definition

A second area which merits investigation in an HOV priority lane system is the concept of a spatially varying car pool definition. As has been mentioned, it is quite possible that within a given priority lane system, sections with distinctly different demand/capacity ratios may be evident as a result of demand variations, capacity variations, or both. If this situation does exist, it could possibly be used to an advantage in lowering the overall travel time by developing different car pool requirements for the various sections.

In considering this approach, a series of analyses with the car pool definition model would be of use. The potential benefits of this concept can be determined by performing an independent analysis for each system section in which the capacities, demands, or operating characteristics vary. These applications would then define the optimal car pool definition for each section, which in turn would become a set of spatially varying car pool definitions for the entire system.

Another application of this concept might also be of interest in conjunction with priority lane systems. This is the determination of the appropriate geographical limits for the priority treatment. Conceptually, this determination can be made by extending the idea of a spatially varying car pool definition to its logical conclusion. This would occur at the point where the optimal definition is found to be 1 person per vehicle. With this definition, the priority lane no longer offers any benefits to high-occupancy vehicles and should be discontinued. Thus, with this technique, the point at which the lane should be returned to normal operations can be determined. The point at which the priority treatment should begin can be similarly defined as that point at which the optimal car pool definition becomes 2 or more persons per vehicle.

Temporal Variation of the Car Pool Definition

A third area of consideration with regard to priority lane systems is the use of a temporally varying car pool definition. Inherent in the structure of the analysis model previously developed is the assumption of constant levels of demand. While this was expedient for the modeling process, it does represent a simplification of reality, in that the peak-period demand pattern is normally triangular or trapezoidal with respect to time. By investigating this demand fluctuation, it might be possible to improve the system operation by developing a time variant set of car pool definitions.

The analysis procedure required for this investigation can be described as the development of an independent car pool definition

for each of several consecutive time blocks within the period of peak flow. These blocks or "time slices" might normally be in the 15 to 30 minute range. Within each time slice, the procedures outlined for the development of a basic car pool definition would be applied, and the resulting set of definitions would define the optimal car pool requirements as a function of time within the peak period.

Additionally, the application of this concept can be used to determine the time period during which the lane should be operated as a priority lane. This determination can be made by extending the temporal variation analysis to determine the time before the peak period at which the optimal car pool definition becomes greater than 1 person per vehicle, and the time after the peak at which the definition falls to 1 person per vehicle. At those times, the lane should commence priority operations or return to normal operations.

General Guidelines

Objectives

The car pool definition model can also be used to develop some general guidelines as to the effects of varying operating conditions on the total travel time. In this final section, these considerations will be addressed. Specifically, the effects of varying violation and nonutilization rates, and the impact of the priority allocation philosophy as reflected by the 3 levels of priority constraints will be investigated.

Test System

The system selected for use in the development of these general guidelines is a freeway section 1 mile in length, with the total number of lanes varying from 3 to 5. In all 3 cases, it was assumed that 1 lane was reserved for priority operations and that the previously developed demand-speed relationships were applicable to both sections. The capacities of the reserved and nonreserved sections were assumed to be 1,500 and 2,000 vehicles per hour per lane, respectively, and the demand distribution presented in Table 4.2 was used to determine the demand distributions for each configuration.

Violation Rate Variation

The first parameter to be considered is the rate at which non-priority vehicles use the priority lane in violation of the imposed restriction on minimum occupancy. This violation rate was allowed to vary from 0 to 25%, at an overall demand to capacity (D/C) ratio of 1.0 for all configurations. The results of the subsequent analyses, shown in Figure 4.9, demonstrate that while the total vehicle-hours of travel were unaffected, the total passenger-hours were increased as the violation rate increased. These results could be anticipated since the violation of the reserved lane is, in essence, a one-to-one exchange of priority and nonpriority vehicles. This exchange keeps the total vehicular demand in each section constant at optimality, so that the total vehicle-hours of travel are unchanged. It does not, however, maintain the same total passenger flows; rather it trades a vehicle at a low occupancy for one at a high occupancy. This net

increase in the number of persons traveling in the lower speed, non-priority section results in increased total passenger travel times under optimum conditions. Additionally, Figure 4.9 also shows that the impact of increased violation rates is more severe for systems with lower net capacities.

Nonutilization Rate Variation

Another parameter to be examined is the nonutilization rate. This is the rate at which qualified priority vehicles choose not to use the reserved section and travel in the nonreserved lanes. In this investigation, the nonutilization rate was also varied from 0 to 25%, at an overall D/C ratio of 1.0. The results of these analyses are shown in Figure 4.10 and are quite similar to those demonstrated for the violation rate variation. This would be expected, inasmuch as the nonutilization rate is effectively the priority section counterpart of the violation rate, and results in an identical one-to-one exchange of vehicles. The decreased magnitude of the nonutilization rate impact results from the relatively low number of priority vehicles requiring less net passenger exchange at a given level.

Level of Priority Variation

The third area to be investigated is the effect of the amount of priority given to the high-occupancy vehicles. In the proposed model, the level of priority is defined by the relative D/C ratios for the 2 sections, and 3 different techniques for allocating this priority are available. These techniques are classified as fixed level of priority, increasing priority, and decreasing priority. In

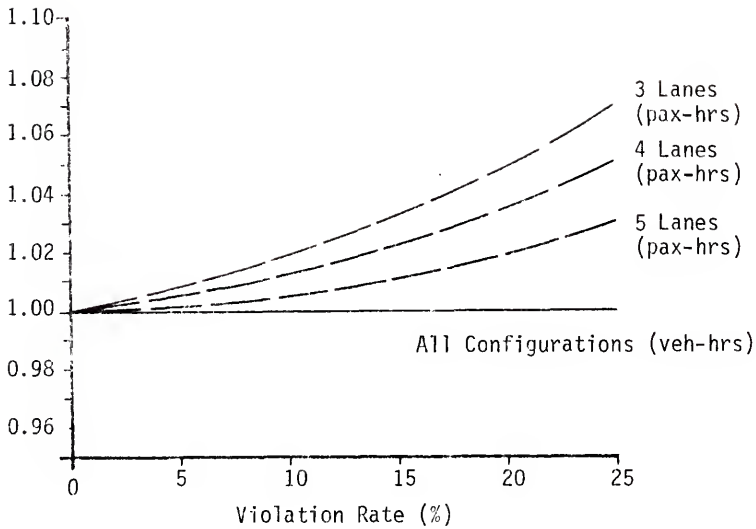


Figure 4.9 THE EFFECT OF VIOLATION RATE ON TOTAL TRAVEL TIME

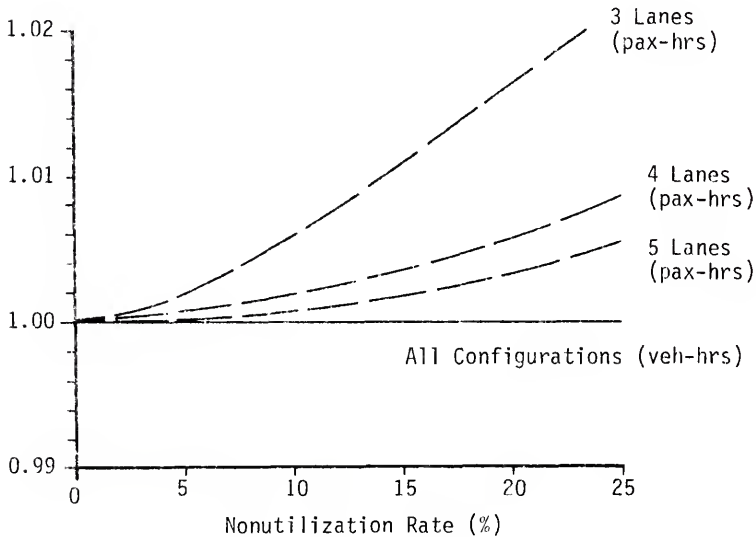


Figure 4.10 THE EFFECT OF NONUTILIZATION RATE ON TOTAL TRAVEL TIME

this investigation, it is necessary to consider these alternative techniques for priority assignment individually.

The first technique, fixed level of priority, requires that the priority section D/C ratio be some constant amount below that of the nonpriority section, as expressed in Equation 4.2.

$$D/C_n - D/C_p \geq \alpha \quad (4.2)$$

The parameter α then determines the level of priority which is to be given to priority vehicles. This constant was varied over its practical range of 0 to 1, with an overall D/C ratio of 1.0, and the results of the analyses for a 3-lane facility are shown in Figure 4.11. This figure shows that as this parameter increases, the total travel time is decreased in the priority section and increased for both the nonpriority section and the total system. Similar analyses show that this same effect can be expected, although at a lesser magnitude, for both 4- and 5-lane facilities

The second priority level allocation technique, increasing level of priority, assigns increasing priority as the system demand increases, as expressed in Equation 4.3.

$$\beta(D/C_n) - D/C_p \geq 0 \quad (4.3)$$

The parameter β then determines the degree of priority for high-occupancy vehicles at any demand level. The effect of variations of this constant over the range 0 to 1 was investigated for the 3 classes of facilities, and the results for the 3-lane system are

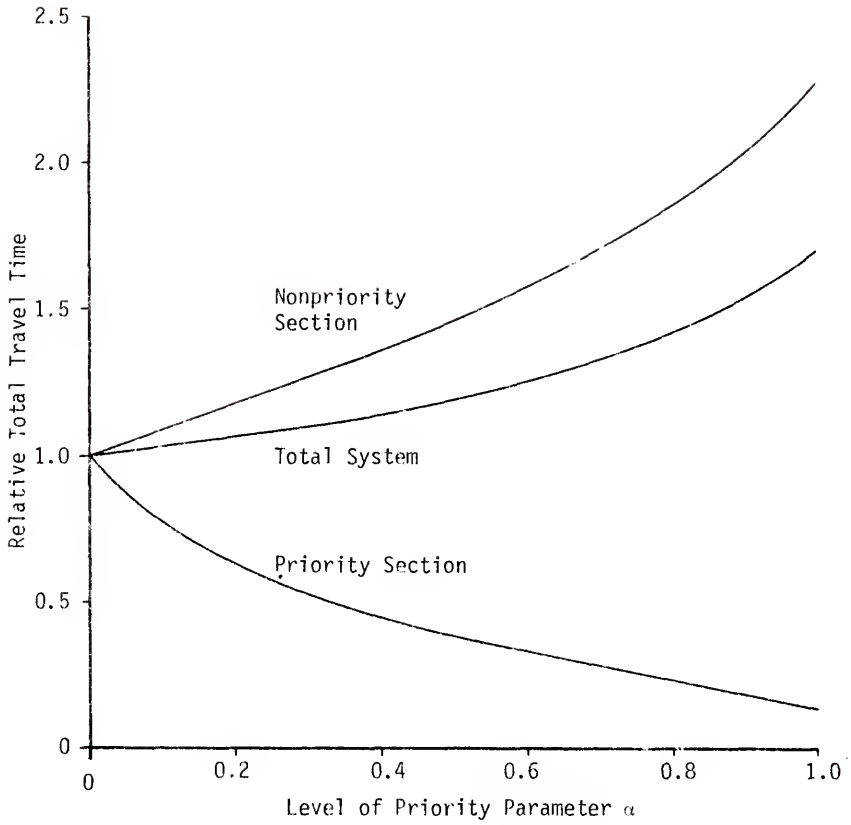


Figure 4.11 THE EFFECT OF A FIXED LEVEL OF PRIORITY ON TOTAL TRAVEL TIME

presented in Figure 4.12. This figure shows that the priority section total travel time is decreased in a nearly linear fashion as more priority is assigned, and that the nonpriority section and total system total travel times are nonlinearly increased. The analyses for the 4- and 5-lane systems also show similar effects, although again at a lower magnitude.

The final priority assignment technique is one of decreasing level of priority. The strategy, Equation 4.4, gives lower priority to the reserved section flow as the overall system capacity is approached.

$$\frac{1}{1-\rho} \cdot (D/C_n) - D/C_p \geq \frac{\rho}{1-\rho} \quad (4.4)$$

This priority determination is controlled by the value of the parameter ρ and the total system demand. The effect of variation of this constant on a 3-lane facility operating at an overall D/C ratio of 0.8 is shown in Figure 4.13. Similar results were obtained for both the 4- and 5-lane facilities, although the magnitude of these effects were again lessened in both cases.

Summary

In summary, these investigations have shown the effects of several parameters on the operation of a priority lane system. It was found that variations in the violation and nonutilization rates have adverse effects on minimum passenger-hours of travel, but do not alter the minimum vehicle-hours. These effects were shown to be more pronounced for systems with lower total capacities. It was also found that the level of priority constraints are an effective means by which the degree of preferential treatment can be controlled.

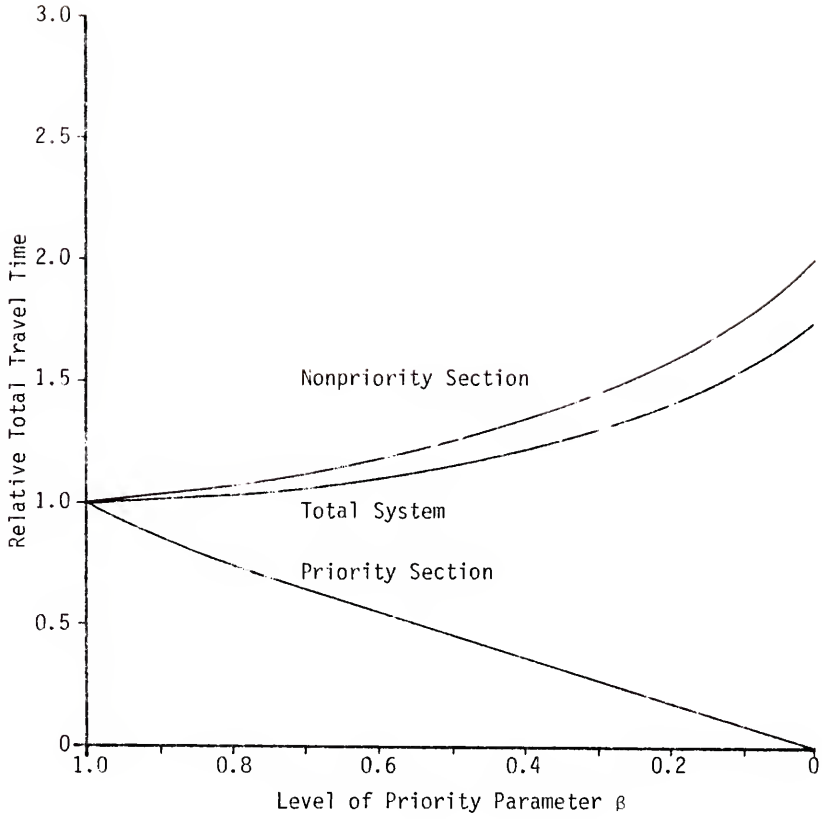


Figure 4.12 THE EFFECT OF AN INCREASING LEVEL OF PRIORITY ON TOTAL TRAVEL TIME

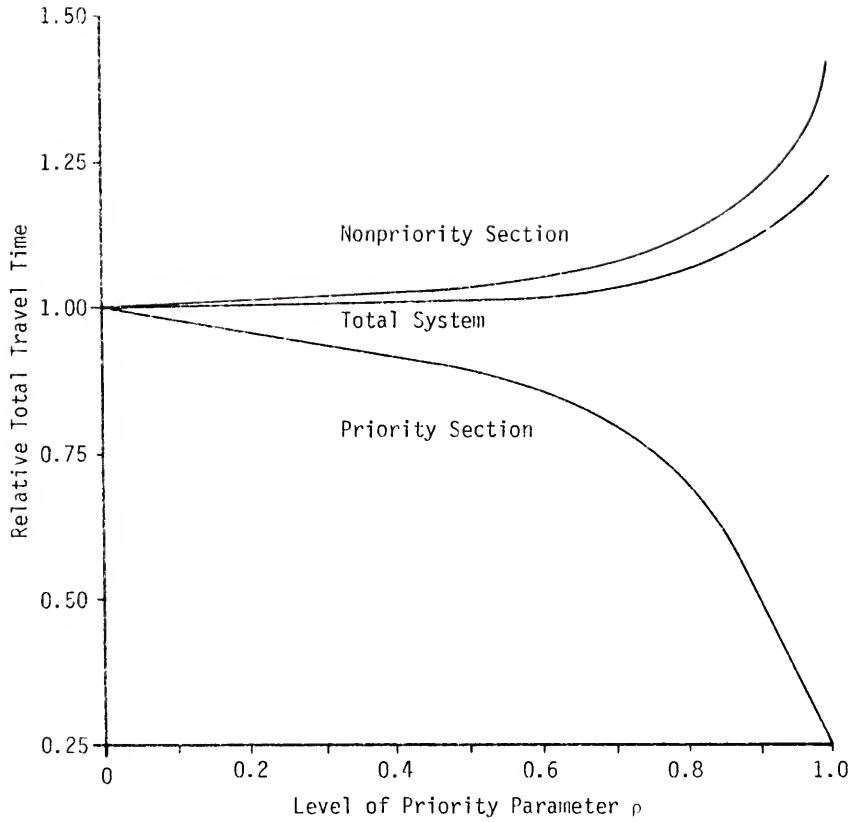


Figure 4.13 THE EFFECT OF A DECREASING LEVEL OF PRIORITY ON TOTAL TRAVEL TIME

CHAPTER 5

DEVELOPMENT OF A PRIORITY LANE ENTRY/EXIT MODEL

Introduction

Objectives

Another consideration in the design and operation of a high-occupancy vehicle (HOV) reserved lane system is the manner in which priority lane access and egress is to be provided. For any implementation of this priority concept, a decision must be made, either by choice or default, as to what entry/exit strategy will be utilized. As was the case with the car pool definition, this should be based on an engineering analysis of each particular situation. As a practical matter, however, this choice has also been based on engineering judgment or social and political considerations in the past. One of the primary reasons for this situation is again the current lack of an appropriate, general purpose analytical tool by which the "best" entry/exit strategy can be determined for a given system.

In this chapter an examination of the alternative priority lane entry/exit strategies will be presented. After this, a methodology for determining the "best" strategy for a particular system will be developed. This development will consist of reducing the physical priority lane system operation to a mathematical basis and introducing a technique which will determine the optimal entry/exit operation. It

is intended that the end product of this development will be a rational procedure, in the form of a mathematical decision-making model, for determining the entry/exit strategy which will provide the best level of operation for an HOV priority lane system.

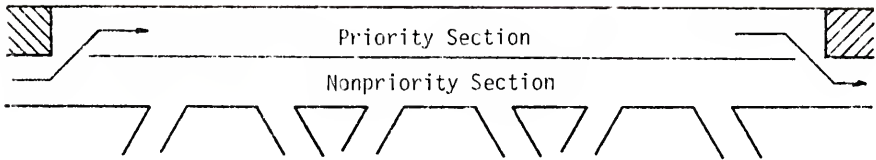
Description of the Problem

Prior to the development of this entry/exit model, some comments regarding the nature of the system to be considered are in order.

First, as with the car pool definition model, the technique to be developed will be primarily oriented toward freeway applications. For this application, three alternative entry/exit strategies can be identified. The most restrictive strategy is one in which entry to the priority section is provided only at the upstream end of the system, and exit is provided only at the downstream end. This strategy is termed "end point" entry/exit.

On the other hand, the least restrictive operation allows entry or exit at any point along the length of the priority section. This strategy can be described as "continuous" entry/exit.

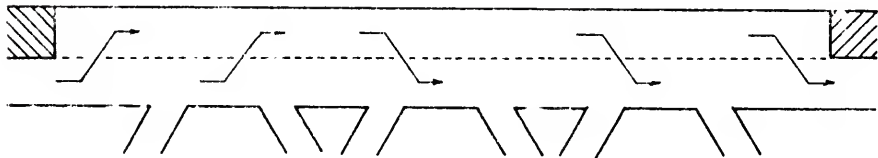
Bridging the area between these extremes is the "discrete" entry/exit strategy. This approach provides points of access and egress at selected locations along the length of the priority lane. As these points become more frequent, the entry/exit operation resembles a continuous strategy, and as they become less frequent, the discrete strategy operates much like the end point strategy. These alternative strategies are shown in Figure 5.1, and a summary of their relative advantages and disadvantages is presented in Table 5.1.



End Point Entry/Exit Strategy



Discrete Entry/Exit Strategy



Continuous Entry/Exit Strategy

Figure 5.1 ALTERNATIVE PRIORITY LANE ENTRY/EXIT STRATEGIES

Table 5.1 SUMMARY OF ALTERNATIVE ENTRY/EXIT STRATEGY CHARACTERISTICS

Characteristic	End Point Entry/Exit	Discrete Entry/Exit	Continuous Entry/Exit
Implementation : Cost Time	High Substantial	High Substantial	Low Minimal
Integrity of Priority Lane Operation	Assured	Varies	Minimal
Maximum Use of Priority Lane Capacity	Doubtful	Varies	Possible
Off Peak Priority Lane Utilization	Difficult	Doubtful	Possible
Adjustment for Variations in Demand	None	Long-Term Only	Self-Adjusting
User Information Requirements	Minimal	Varies	Substantial
Enforcement Requirements	Lessened with Proper Design	Varies	Substantial
Associated Weaving Problems	Lessened with Proper Design	Varies	Substantial
Accident Potential	Least	Varies	Greatest

In addition to the previous restriction to freeway applications, it will be assumed that the priority system has been defined with respect to the car pool definition. This assumption will permit treatment of the demand in terms of priority or nonpriority vehicles rather than by individual level of occupancy. However, as will be discussed in the following chapter, the technique to be developed can be used in this selection process under certain conditions. It will also be assumed that the length of the priority system and the number of reserved and nonreserved lanes has been established.

Thus, within these limitations and assumptions, an entry/exit strategy analysis technique will be developed as a decision-making process. The resulting recommendations will not represent an operational control strategy per se, but will determine the best system configuration under which optimal performance can be achieved.

Method of Analysis

Previous research in the area of HOV priority techniques has not produced a direct technique for investigating priority lane entry/exit strategies. As discussed earlier, the development of general purpose analysis techniques for HOV treatments has been limited to the area of operational simulation tools. For freeway systems, the most notable of these are the PRIFRE and FREQ3CP models.

The PRIFRE model is a simulation package for reserved freeway lane operations which is primarily oriented toward the evaluation of priority lane operations for user-selected control strategies and utilizes the end point entry/exit strategy as a basis for operation.

Through manual interface procedures this model can be used to approximate a discrete entry/exit strategy [Minister et al., 1973].

The FREQ3CP model was developed as an analytical tool primarily for use in investigating priority entry techniques. Although this model is a simulation process, some decision-making capability is included for priority entry considerations. The use of a reserved lane for high-occupancy vehicles is included as a simulation option within this model. Although the simulation assumes a continuous entry/exit strategy, the discrete strategy can be approximated with specified procedures [Ovaici et al., 1975].

Thus, the existing techniques for investigating the priority lane entry/exit strategies are limited to simulation models. Through judicious application of these models, alternative strategies can be evaluated. However, the best strategy can only be determined through an exhaustive search procedure.

In considering the approach to be taken in the development of this entry/exit analysis model, several candidate techniques were examined. These were: (1) analytical models, (2) simulation models, and (3) optimization models.

Looking first at the analytical approach, this methodology would consist of developing an equation or set of equations which would describe the physical system mathematically and could be solved directly to determine the best strategy for a particular implementation. This technique would obviously be the most direct approach if an analytical solution exists. Considering the complexity of the system, the introduction of the human element in the form of motorists, and

the empirical nature of many of the traffic flow relationships, it is unlikely that a direct analytical solution could be formulated. If one should exist, it would likely be quite complex and unwieldy.

An attempt to develop a decision-making technique with a simulation process would avoid the potential unwieldiness of an analytical solution, and, as demonstrated by the previous research, could result in a workable procedure. The simplest approach at this point would be the adaptation of existing simulation models to a search procedure which would allow selection of the best alternative from a number of candidate strategies. While this approach would be based on proven techniques and is conceptually easy to grasp, it is estimated that for a priority lane system that is evaluated with 10 subsections (entry or exit points), over 900 possible entry/exit combinations would require evaluation. This methodology can be classified as an "exhaustive search" and would be highly inefficient.

The final category of techniques to be considered is the area of optimization models. As was discussed in conjunction with the car pool definition model, this approach utilizes a mathematical description of the physical system in the form of an objective function which is to be optimized (maximized or minimized) which is subject to a set of constraint equations. The optimization technique then determines that combination of decision variables which will result in the optimum value of the objective function within the solution limitations imposed by the constraint equations [Hadley, 1963, p. 1]. Thus, this approach would require a mathematical description of the system similar to those

required by the previous techniques and would provide a direct determination of the optimal entry/exit configuration within the constraint limitations.

With this discussion of the advantages and disadvantages of the candidate solution techniques, an examination of the desirable qualities of the selected approach is in order. Briefly, these considerations are as follows:

1. The model should adequately represent the physical system.
2. The model should be sufficiently general for application to a reasonable range of situations.
3. The model should, to the extent possible, be adaptable to a variety of special considerations.
4. The model should produce as a final result a recommended entry/exit strategy for each application.

Based on these desirable qualities, the candidate approaches can again be considered. An analytical approach might possibly be formulated in such a manner as to adequately reflect the physical system and produce a recommended strategy. However, this approach, if indeed possible, would be quite limited in terms of flexibility and adaptability.

The simulation technique would result in a model that is representative of the true system operation, general-purpose in scope, and reasonably adaptable to special conditions. Still, a pure simulation approach would not produce a recommended strategy unless it were

coupled with an exhaustive search technique. This combination tends to be highly inefficient for realistically large systems.

A methodology based on an optimization technique could be developed in such a way as to adequately approximate the physical system operations and produce as the final product a recommended entry/exit strategy. Additionally, this methodology would be applicable to a variety of situations, and, to some extent, would be adaptable for special conditions.

Based on the previous work in this area, the advantages and disadvantages of the candidate techniques, and the desirable qualities of the final product, it is felt that an optimization approach would be most satisfactory for this development. In the remainder of this chapter, additional modeling considerations will be presented and an entry/exit strategy optimization model will be developed.

Development of the Model

Objective

Within the framework of the preceding discussion, the objective for this investigation can be restated as the development of an optimization model for determining the optimal entry/exit strategy for a freeway-based HOV priority lane treatment. The considerations to be addressed in this section include the general model structure, the mathematical development of the model in terms of its objective function and constraint equations, and the physical implications derived from this mathematical structure.

General Structure

As was previously discussed, the physical system to be addressed is a section of directional freeway through which a portion of the traffic lanes have been designated for the exclusive use of high-occupancy vehicles. The length of this priority section has been predetermined, as well as the number of lanes to be reserved for use by priority vehicles. The vehicular demand enters this section from the upstream freeway mainline or the entrance ramps along the length of the roadway. Vehicles can exit the system on either the downstream mainline or exit ramps along its length. This demand can be stratified as priority or nonpriority vehicles on the basis of a previous high-occupancy vehicle definition. The system has known vehicular capacities in both the reserved and nonreserved sections which are available to accommodate the demands. The speed-flow, or demand, characteristics of the two sections are independent and predictable.

The intended operation of this system is for the nonpriority demand to enter the freeway and remain in the nonreserved section for the trip through the system, whereas, the priority vehicles enter the freeway and proceed through the system in the reserved lane(s). Although the nonpriority vehicles are restricted to use of the nonreserved section only, the priority traffic can elect to travel in either section as desired. In a properly designed system it is anticipated that travel in the reserved section will result in a decrease in the trip time for priority vehicles, however, the individual motorists have the option to enter and exit the priority lane at their discretion.

Conceptually, this system can be divided into a series of subsystems which are homogeneous with respect to the reserved and non-reserved section capacities, demands, and speed-flow characteristics. Each subsystem has a reserved section and a nonreserved section. The nonpriority demand movements in each subsystem are limited to travel in the nonreserved section and freeway entry or exit maneuvers. The priority vehicles can, at the upstream end of any subsystem, elect to enter or exit the reserved section and/or the freeway proper. Once this decision is made, travel through the subsystem will be in the selected section. It is reasonable to assume that since the characteristics of each subsystem are invariant, reserved/nonreserved system interchanges would not be desirable within any one subsection. This conceptual operation is shown in Figure 5.2.

Mathematical Development

At this point the optimization model can be fully developed in such a manner as to reflect the conceptual system operation as described and to determine the preferable entry/exit configuration for the reserved lane treatment. The first step in this development is the identification of the basic model structure. Next, the objective function can be developed in detail, followed by consideration of the required constraints. With this completed, the practical implications of the constraint equations will be examined.

In the area of mathematical optimization, several techniques are available for consideration, as was discussed in conjunction with the car pool model. These range from classical methods, such as the

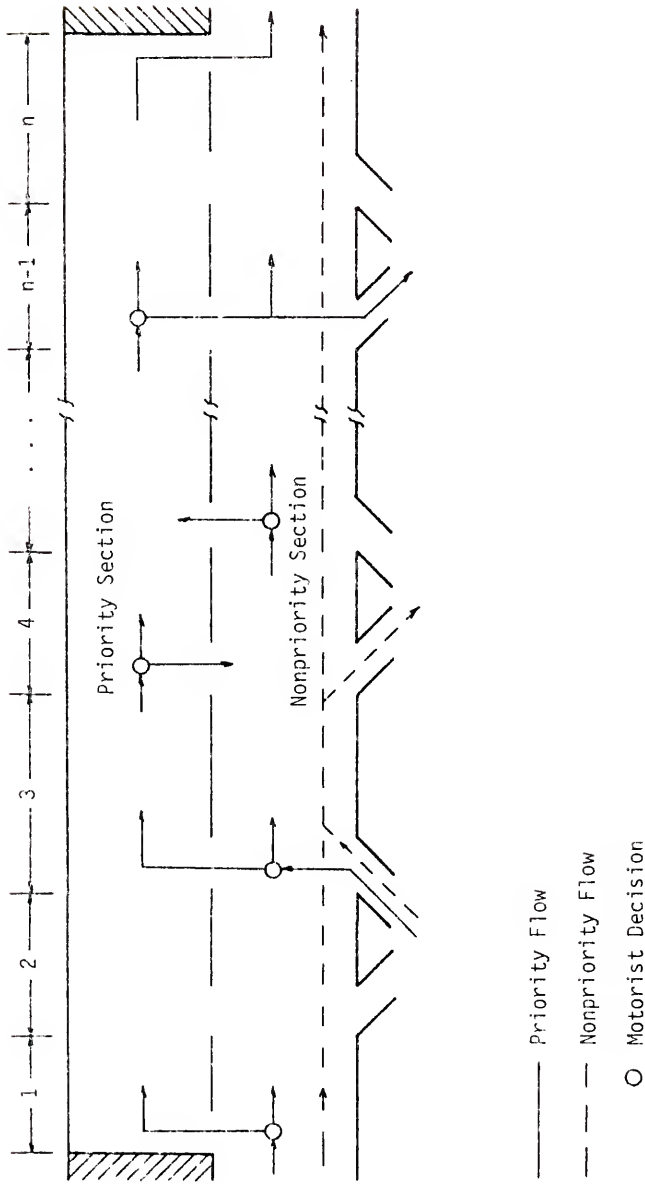


Figure 5.2 CONCEPTUAL MODEL OF PRIORITY LANE SYSTEM OPERATION

calculus of variations, to more advanced methods, such as dynamic and geometric programming, with each method having a range of extended techniques [Sivazlian and Stevens, 1975]. An efficient method of addressing this particular problem can be identified within the general area of linear programming. Specifically, an optimization technique based on network flow theory, often referred to as graph theory, can be applied to the development of an entry/exit strategy optimization procedure.

The general structure of network flow structure can be summarized as follows. A network or linear graph can be thought of as a finite set of points $N=\{n_1, n_2, n_3, \dots, n_m\}$, and a finite set of lines which are ordered pairs of the elements of N , $A=\{(n_1, n_2), (n_1, n_3), \dots, (n_{m-1}, n_m)\}$. The elements of N , variously referred to as nodes, vertices, or points, normally represent physical locations such as origins or destinations. The elements of A , known as lines, arcs, edges, or branches, then represent the set of flow paths between the various nodes. This then defines a directed network of linear graph $G=(N;A)$ [Ford and Fulkerson, 1962, p. 2]. This network can be pictured by defining a point for each $n_x \in N$, and drawing an arrow or arc from n_x to n_y if the ordered pair (n_x, n_y) is an element of A . The full network will be represented when $x=(1,2,3,\dots,n)$ and $y=(1,2,3,\dots,n)$. For example, the network shown in Figure 5.3 consists of five nodes 1, 2, 3, 4, 5, and 8 arcs $(1,2)$, $(2,3)$, $(3,5)$, $(4,1)$, $(1,4)$, $(4,3)$, $(5,2)$, $(5,4)$.

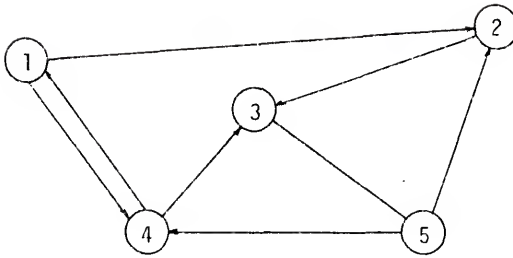


Figure 5.3 NETWORK FLOW STRUCTURE

In some cases, the network might be undirected or have mixed directed and undirected arcs. It is also possible to have arcs (n_x, n_x) such that the node receives flow from itself. For this application, however, these undirected and feedback arcs are not required for the problem formulation and can be neglected.

The particular type of network to be utilized in this development is known as a circulation network. A circulation network is an extension of the general structure in that it is required that the flows have both an origin and a destination, called the source and sink, respectively [Sivazlian and Stevens, 1975, p. 240]. This extended structure has a definite mathematical formulation and, as such, can be solved with specialized solution techniques. This mathematical structure is expressed in Equations 5.1 through 5.4 [Wagner, 1975, p. 965].

$$\text{Minimize } Z = \sum_{i=1}^n \sum_{j=1}^n c_{ij} x_{ij} \quad (5.1)$$

$$\text{Subject to: (1) } L_{ij} \leq x_{ij} \leq U_{ij} \quad (5.2)$$

$$L_{ij} \geq 0$$

$$U_{ij} \leq +\infty.$$

$$(2) \quad \sum_{i=1}^n x_{ij} - \sum_{k=1}^n x_{jk} = 0 \quad (5.3)$$

$$\text{for } j=(1,2,3,\dots,n)$$

$$(3) \quad x_{ij} \in \{0,1,2,\dots,\infty\} \quad (5.4)$$

Where:

- c_{ij} = unit flow cost on arc ij
- x_{ij} = flow on arc ij
- L_{ij} = lower flow limit for arc ij
- U_{ij} = upper flow limit for arc ij .

These mathematical expressions define a linear programming model in which the objective is to minimize the total flow cost subject to restrictions on the amount of flow to be assigned to any arc, the disallowance of storage at any node, and the requirement of integer flow assignments. The flow level restriction defines an upper and lower bound on the flow assigned to a given arc. This lower bound can conveniently be thought of as the minimum flow requirement on the arc, and the upper bound can be characterized as the capacity of the arc. The second constraint, Equation 5.3, requires that the net storage at a given node be zero. That is to say that the total flow out of a node must be equal to the total flow into the node. Finally, the last constraint requires that the assigned flows be integer quantities.

Thus, for a circulation network formulation, there is associated with each flow arc a nonnegative flow requirement (L_{ij}), a positive flow capacity (U_{ij}), and a unit flow cost (c_{ij}). No storage is allowed at any node, and all assigned arc flows must be integer quantities. The origin of all network flow is the source node, and the destination is the sink node. For certain conditions, the source and sink nodes can be combined into a single source/sink node.

Recalling the conceptual physical system presented in Figure 5.2, this network structure can then be adapted to describe the priority treatment operation. This representation is shown in Figure 5.4. In this structure, node 1 is the combined source/sink node, nodes 2,4,6,...,2n represent the upstream end of the nonpriority section for each of the n subsections, and nodes 3,5,7,...,2n+1 are the upstream ends of the priority sections. Subsystem n+1 (nodes 2n+2 and 2n+3) represents the downstream terminus of the system. The arcs associated with node 1 represent the freeway entry and exit points, mainline and intermediate ramps, while the horizontal arc chains are the priority and nonpriority flow sections connecting adjacent nodes. Finally, the vertical arcs represent the flow interchange between the reserved and nonreserved section at the upstream end of each subsystem. By assigning the system demand to this network in such a manner as to minimize the total flow cost, the optimal entry/exit strategy can be defined as the set of interchange points between the priority and nonpriority sections with nonzero flow assignments.

With the general model structure selected and the representation of the physical system established, the mathematical model may

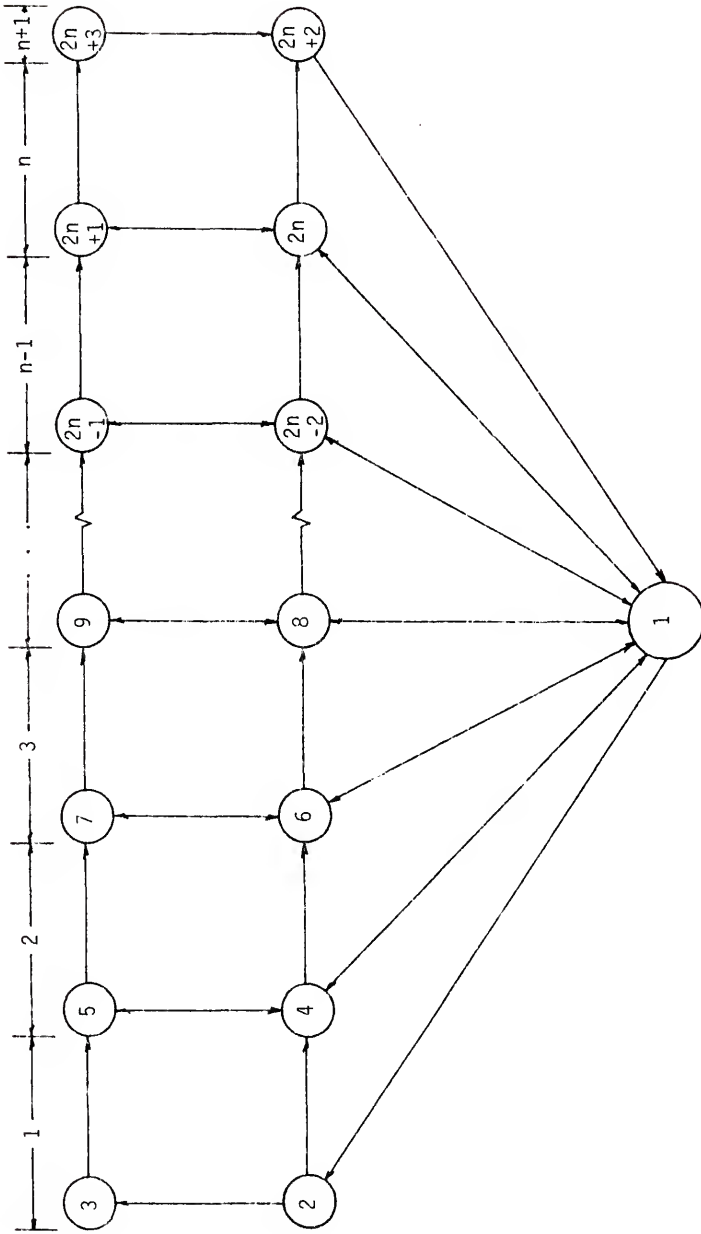


Figure 5.4 NETWORK MODEL OF PRIORITY SYSTEM FLOW

now be developed in detail. First, the objective function will be fully formulated, then the model constraints will be related to the physical system parameters. Finally, the implications of this development with respect to the physical operation will be discussed.

As stated previously, the objective of this model is to minimize the total flow cost for the network. The initial step in fully defining this objective is the selection of the figure of merit to be considered. Traditionally, several measures have been used to measure the quality of travel within a system. These include such criteria as total travel time (both passenger and vehicle), average trip time, delay, and operating speed. For this investigation, minimization of the total vehicular travel time was selected. This was based on consideration of the fact that the selected entry/exit strategy is not a positive control system; rather it is a reflection of how the priority lane should be utilized in order to achieve optimal performance. The priority vehicle drivers are allowed to determine the manner in which they utilize the lane; thus the selected measure should be indicative of their decision-making process.

In his classic paper, Wardrop [1952] proposed two principles concerning the route selection process by regular users through a system. These principles state that regular users will distribute themselves in such a manner that:

1. The journey times on all routes actually used are equal, and less than those which would be experienced by a single vehicle on any unused route.
2. The average journey time is a minimum.

Since diversion of demand will not be considered in this model, these principles will be reflected by the minimization of the total travel time within the system.

Having selected the measure of effectiveness to be minimized, the objective function for this model can be restated as shown in Equation 5.5.

$$\text{Minimize } TTT = \sum_{i=1}^n \sum_{j=1}^n T_{ij} \cdot x_{ij} \quad (5.5)$$

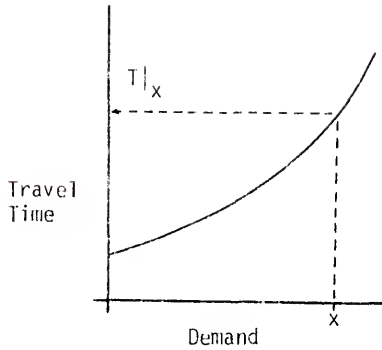
Where: T_{ij} = unit travel time on arc ij

x_{ij} = flow on arc ij .

This restatement expressed the total travel time as a function of the unit travel time and flow. However, as discussed previously, the unit travel time on a roadway is a function of the flow. This relationship causes the objective function as stated in Equation 5.5 to be nonlinear, which violates the requirements of the general linear programming structure.

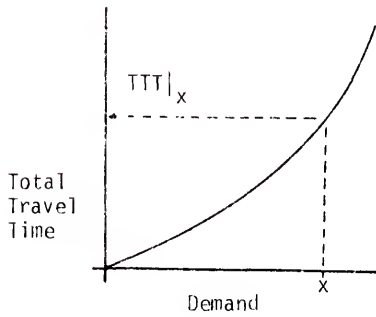
As was demonstrated in the development of the car pool definition model, this irregularity can be removed with a reasonably simple technique. First, by recognizing that the total travel time at a given demand level x is defined as $TTT|_x = x \cdot T|_x$, the objective figure of merit can be expressed as a direct function of the demand. This is shown in Figures 5.5a and 5.5b.

However, this does not eliminate all problems associated with the objective function in that the total travel time vs. demand



$$TTT|_x = x \cdot T|_x$$

(a) Travel Time vs. Demand



$$TTT = f(\text{Demand})$$

(b) Total Travel Time vs. Demand

Figure 5.5 EVOLUTION OF A TOTAL TRAVEL TIME VS. DEMAND RELATIONSHIP

relationship is itself nonlinear. A classical method, as discussed for the car pool definition model, for resolving this nonlinearity is the use of a series of line segments to approximate the curvilinear relationship. The resulting set of line segments is known as a piecewise linear approximation (PLA) [Wagner, 1975, p. 563]. This technique is demonstrated in Figure 5.6. The use of this approximation has the effect of treating each flow path as a set of flow branches, one for each line segment, with each having a fixed flow cost and capacity. This branching effect is shown in Figure 5.7.

With these revisions, the objective function can be restated as a linear function as shown in Equation 5.6.

$$\text{Minimize TTT } \sum_{i=1}^n \sum_{j=1}^n \sum_{k=1}^m S_{ijk} \cdot x_{ijk} \quad (5.6)$$

Where: S_{ijk} = unit flow cost on branch k or arc ij
 x_{ijk} = flow on branch k or arc ij.

This final formulation can then be used as the objective function of the flow optimization model.

With the objective function fully defined, the model constraints, presented in Equations 5.2 through 5.4, can be considered in detail. These constraints require that the assigned flow on any arc be within certain limits, that the net storage at any node be equal to zero, and that the flow assignments be integer quantities.

The flow limits constraints, Equations 5.7 and 5.8, serve several functions in the model structure.

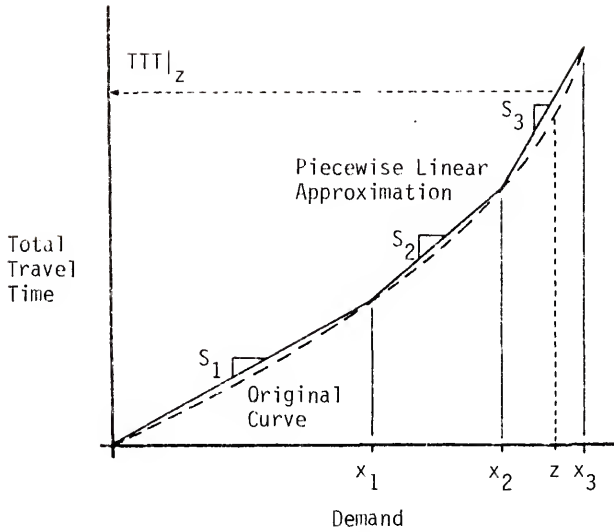


Figure 5.6 PIECEWISE LINEAR APPROXIMATION OF TOTAL TRAVEL TIME VS. DEMAND

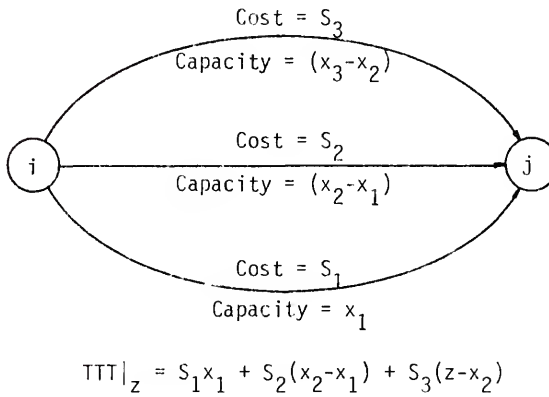


Figure 5.7 EFFECT OF PIECEWISE LINEAR APPROXIMATION

$$x_{ij} \geq L_{ij} \quad (i=1,2,3,\dots,n), (j=1,2,3,\dots,n) \quad (5.7)$$

$$L_{ij} \geq 0$$

$$x_{ij} \leq U_{ij} \quad (i=1,2,3,\dots,n), (j=1,2,3,\dots,n) \quad (5.8)$$

$$U_{ij} \leq +\infty$$

For the freeway entry and exit flows, both mainline and ramp, these constraints are used to define the total system input and output. This is achieved by setting $L_{ij}=U_{ij}$ = total flow for these entrances and exits. For the nonpriority flow arcs, the lower flow bound for a given subsystem should be equal to the total nonpriority demand at the upstream end of the subsystem. This insures a level of flow in the nonpriority section which is at least equal to the nonpriority demand. The upper bound of these nonpriority flow arcs can then be set at a level equal to the nonpriority capacity multiplied by the maximum D/C ratio specified in the speed-flow relationship. For practical consideration, this upper bound should be set sufficiently high to allow the flow assignment to be made on the basis of the speed-flow characteristics rather than the capacity values. In the priority section, the lower limits of the flow should be set to zero to avoid requiring use of the priority section unless it provides some travel time benefits to the priority flow. The upper limits of these arcs should be treated as outlined for the nonpriority arcs. Finally, for the priority-nonpriority interchange arcs, which are defined as priority entry or exit points by the arc orientation, the lower flow limit should be zero so as not to require entry or exit unless a travel time benefit is possible. The

upper limit of this flow can either be used to define the maximum weaving capacity for the particular subsystem or the maximum merging capacity for the traffic lanes involved. If these factors are not to be considered, this upper limit may be considered to be infinity.

The flow balance constraints are defined by Equation 5.9.

$$\sum_{i=1}^n x_{ij} - \sum_{k=1}^n x_{jk} = 0 \quad (j=1,2,3,\dots,n) \quad (5.9)$$

These constraints effectively disallow the storage of flow at any point in the system and, as such, do not require adjustment for different applications.

The integer flow constraints are stated in Equation 5.10.

$$x_{ij} \in \{1,2,\dots,\infty\} \quad (i=1,2,3,\dots,n), (j=1,2,3,\dots,n) \quad (5.10)$$

These constraints require that the flow assignments be positive integer values and are also invariant for all applications.

By utilizing the constraints as outlined, several facets of the physical system operation are included. First, by setting the upper and lower limits of the freeway input and output arcs equal to the actual input or output demand, the total system demand is considered, and the demand levels for each subsystem are representative of the actual situation. Next, by utilizing the total nonpriority demand entering a subsection as the lower limit of that nonpriority flow arc, it insures that only the priority flow will be considered for potential operation in the priority section. However, this also allows consideration to be given to the nonpriority demand in the overall optimization process.

This is to say that the system total travel time is optimized rather than just the priority section total travel time. Finally, the proper selection of the upper flow limit for the priority section entry and exit arcs allows inclusion of physical reality with respect to the weaving and merging capacities of the system elements.

Summary of the Model

In summary, a network flow model has been proposed which will determine the optimal entry/exit strategy for an HOV priority lane system on an urban freeway based on consideration of minimum total travel time. This optimization model is constrained in such a fashion as to realistically reflect the operation of the physical system. The full network structure is shown in Figure 5.8. In this figure, the arc flow parameters are presented in abbreviated notation (unit cost, {lower limit, upper limit}). As can be seen, the ramp travel times are not considered in the total travel time determination.

Solution Methodology

Underlying Process

The determination of an optimal priority lane entry/exit strategy is based upon the development and solution of a deterministic optimization model with a network flow formulation. For the model as developed, the following steps should be taken in the determination of a recommended entry/exit strategy:

1. Establish system parameters and operating characteristics. This includes the determination of the priority and nonpriority demands and the reserved

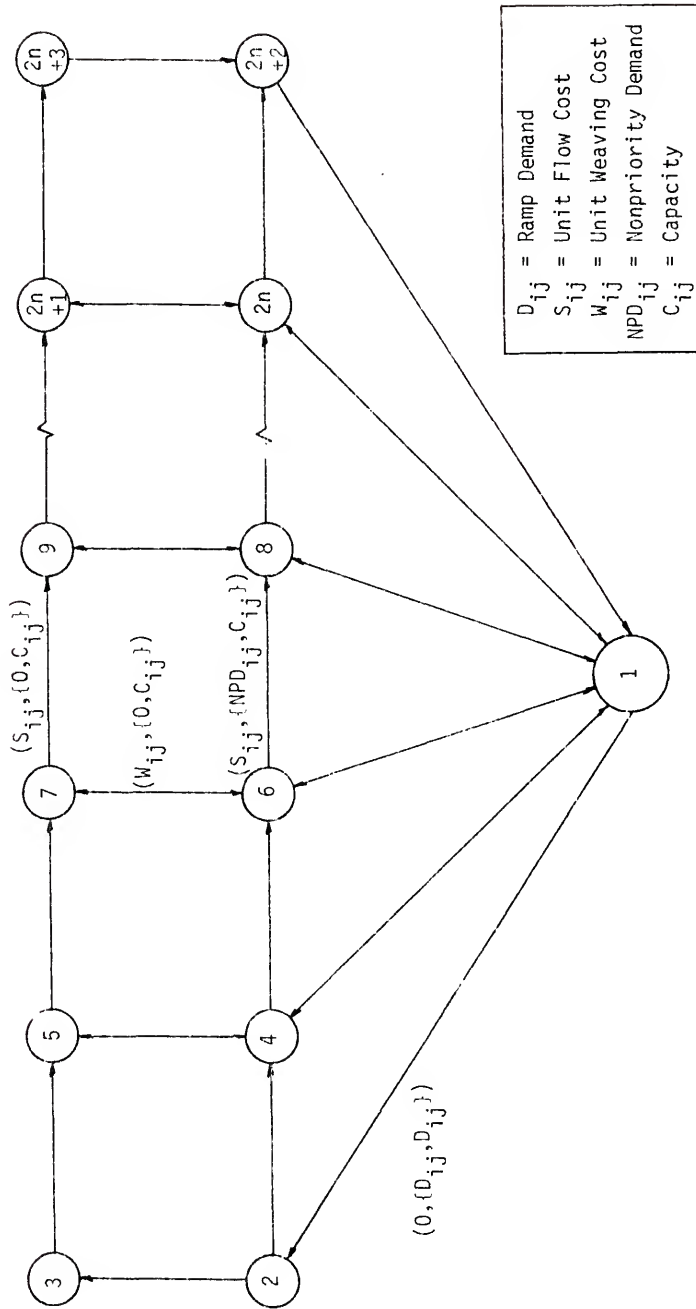


Figure 5.8 FLOW NETWORK FOR THE PRIORITY LANE ENTRY/EXIT MODEL

and nonreserved section flow capacities, speed-flow relationships, and weaving delays for each subsystem.

2. Develop optimization constraints. This involves only the flow limits constraints.
3. Develop cost coefficients for objective function. These are based on the speed-flow relationships and the weaving delay considerations.
4. Optimize the system flow pattern. This is achieved by solution of the network flow model.
5. Formulate recommendations. In this final step, the optimal flow patterns found in the previous step are examined to determine the recommended entry/exit strategy.

This general process is shown in Figure 5.9.

Recommended Techniques

At this point consideration should be given to specific techniques which may be utilized in this solution process. The areas to be addressed include the development of the objective function cost coefficients and the method of solving the network flow model. Although the final model formulation and solution should be based on the particular application and the discretion of the user, the following comments will serve as a set of general guidelines.

In previous discussion, reference has been made to the speed-flow relationships for the priority lane system. Since the model itself does not restrict the system to operating with the demand-to-

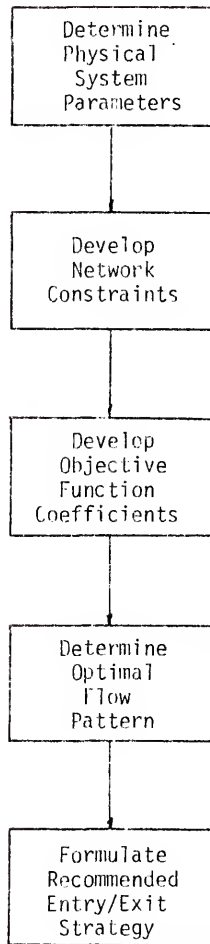


Figure 5.9 GENERAL SOLUTION PROCEDURE FOR THE ENTRY/EXIT MODEL

capacity (D/C) ratio less than 1.0 and these priority treatments are normally considered at a time when the system tends to be congested, it is desirable to consider these relationships in both the under-saturated and oversaturated conditions. Under the first condition, this relationship may be determined by direct field measurement or a general relationship may be obtained from any one of several references. However, in the oversaturated condition this is not possible. In this realm, it is suggested that a travel time estimating technique be employed [Huber et al., 1968]. The use of this method for developing travel time relationships for congested operation will enable the user to combine the relationships for the two operating conditions for use in the entry/exit optimization model. A more detailed discussion of this technique is presented in Appendix A of this report.

A second point to be considered with respect to these relationships is the development of the piecewise linear approximation (PLA) as was discussed in conjunction with the car pool definition model. The use of this approximation technique has the net effect of assuming constant speed operation over the range of each branch of the PLA. The inaccuracies introduced by this can be minimized by the number of line segments used to approximate the actual curve, and by the location of the break points for the PLA. Traditionally, a PLA with three branches has been used to approximate the speed-flow curve for under-saturated operation. Considering the possible extension of this relationship into oversaturated operation, it is suggested that a PLA with four or possibly five segments be developed. Additionally, configuration of the PLA to concentrate relatively short branches about the

expected range of operation will usually improve the overall accuracy of the analysis process.

The final consideration in the development of the objective function cost coefficients is the assignment of costs associated with the priority lane entry or exit maneuver. These costs are not travel times, but rather weaving delay costs. In other words, they should represent the delay experienced, if any, in weaving into or out of the priority lane. In many cases this factor will be negligible. However, in those instances where weaving delays might be experienced, they should be considered. Since the weaving delay is related to the weaving volume, one possible method of treating this cost is the development of a weaving delay vs. flow curve and utilizing the PLA technique as was done with the speed-flow relationship. In assigning these costs, care should be taken to insure that the weaving delays are reasonable and do not negate the potential benefits of the priority lane operation.

With regard to obtaining an optimal solution for the network flow model, several alternative techniques can be considered. Since this model structure is a form of linear programming, the various techniques for solving linear optimization models would also be applicable in this case. However, as a result of the special structure of the network flow model, more efficient techniques are available. The most preferable of these is the out-of-kilter algorithm [Ford and Fulkerson, 1962, pp. 162-169; Wagner, 1975, pp. 965-974]. This technique is reasonably straightforward and readily adaptable for computer applications.

Program Strategy

A final note is that the procedures and techniques previously presented have been used to develop a computerized entry/exit strategy optimization model, STRATEGY. This model will be used to generate solutions for the numerous sample applications to be presented in subsequent chapters of this report.

Since this model is based directly on previous considerations, no documentation of the program operation will be presented here. It should be noted, however, that this program was written in the FORTRAN IV programming language for operation on an IBM System 370/165. A block diagram of the program operation is presented in Figure 5.10 for additional reader information.

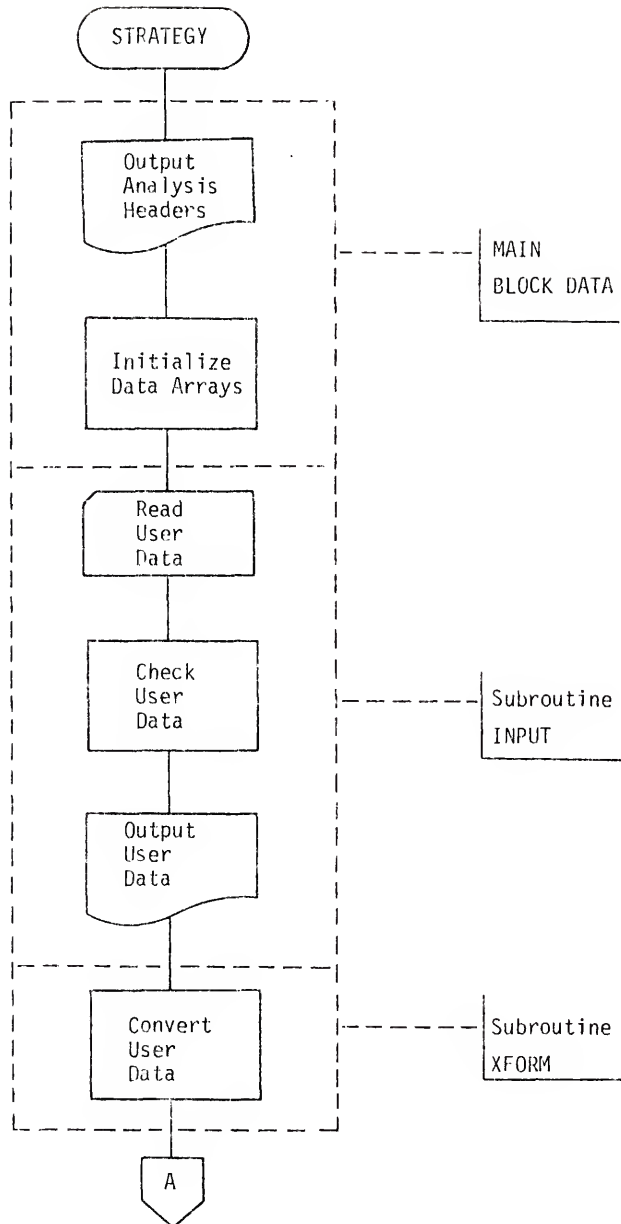


Figure 5.10 BLOCK DIAGRAM FOR PROGRAM STRATEGY

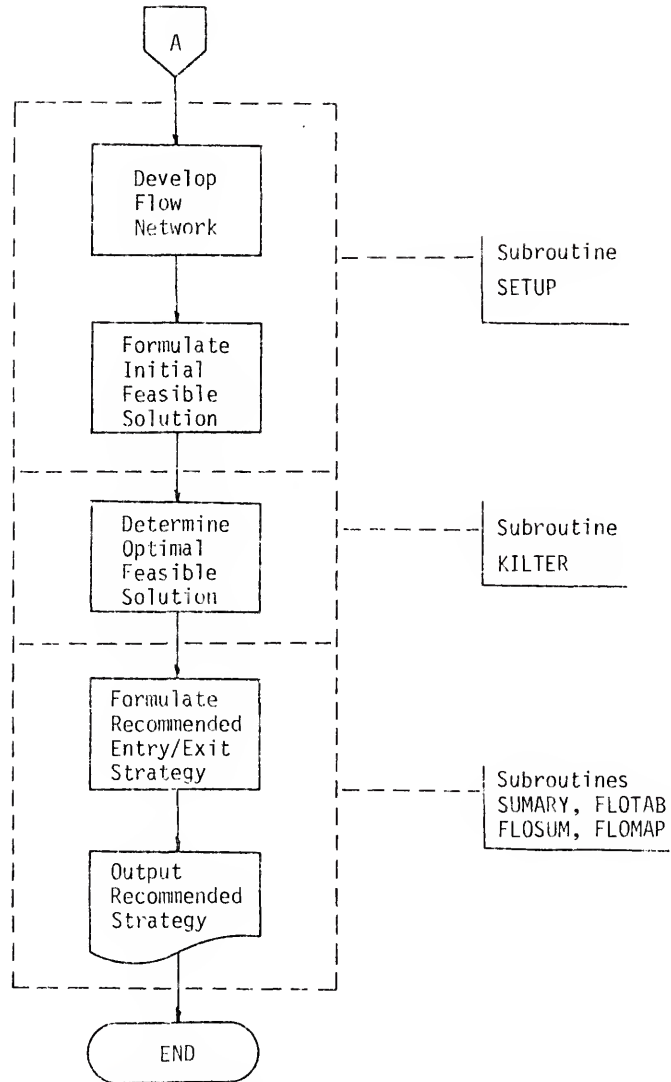


Figure 5.10 - continued

CHAPTER 6

VALIDATION AND APPLICATION OF THE PRIORITY LANE ENTRY/EXIT MODEL

Introduction

Objectives

A primary consideration in the evaluation of any mathematical or physical model is the degree to which the operation of the full-scale system is represented. Unless the model adequately reflects the true system operation, it is not possible to develop meaningful conclusions based on the results of the modeling process. Another important factor in the evaluation of a proposed modeling technique is the flexibility of the model itself. In most cases, a model which has a narrow range of application will be of limited value outside the particular situation for which it was developed.

In this chapter, these considerations will be addressed with respect to the priority lane entry/exit model which has been proposed. The adequacy of the physical system representation will be examined through a model validation exercise, and the flexibility of the model will be demonstrated in a discussion of the potential applications of the model. Additionally, some general observations with regard to the formulation of a priority lane entry/exit strategy will be presented.

Organization

The first area to be examined in this chapter is the validity of the proposed model. This examination will present a number of considerations in support of the accuracy of the modeling process. Next, the flexibility of the model will be discussed in terms of its basic application and additional areas in which the model can be applied will be identified. Finally, some general observations will be presented.

Validation of the Model

Validation Methodology

As was pointed out when considering the validity of the car pool definition model, validation of a mathematical model is the process of verifying that the model accurately reflects the operation of the full-scale system. Traditionally, validation has taken the form of applying the proposed model to a set of characteristics for which the system operation is known, and then comparing the known operation with the results predicted by the model. This process does not necessarily lead to a definitive conclusion with respect to the ability of the model to predict operations under other conditions. However, it has been found to be an acceptable basis for inferring the accuracy of the modeling process.

Inasmuch as the data which are necessary to perform the validation in a traditional manner are not available, and the effort required to produce these data is beyond the scope of this work, a modified approach will be adopted to demonstrate the validity of the

proposed priority lane entry/exit model. The verification of this model will be accomplished in 3 stages: (1) sensitivity analyses, (2) postoptimality analysis, and (3) comparison with current simulation techniques. The sensitivity analyses will demonstrate that basic traffic flow relationships are properly maintained within the model. The postoptimality analysis will establish that the model does, in fact, define optimal system operations. Finally, the accuracy of the proposed technique will be verified through joint application of the optimization model and a simulation model to a series of physical configurations.

Sensitivity Analyses

In this section, a series of analyses will be presented which will demonstrate the sensitivity of the proposed entry/exit model to changes in various system operating characteristics. Additionally, the manner in which certain basic traffic flow relationships are reflected by the model will be identified. Specifically, these analyses will investigate the optimization process under varying priority and non-priority section capacities, and for varying levels of demand. These particular parameters were selected for their underlying roles in the traffic flow process, and because they can be expressed as scalar quantities.

The test system which will be utilized in these analyses is a section of the I-95 HOV priority lane system in Miami, Florida. This test site, shown in Figure 6.1, is a freeway segment 17,190 feet (3.26 miles) in length with 4 nonreserved lanes and 1 reserved lane operating during the peak periods. Alternating along the length of

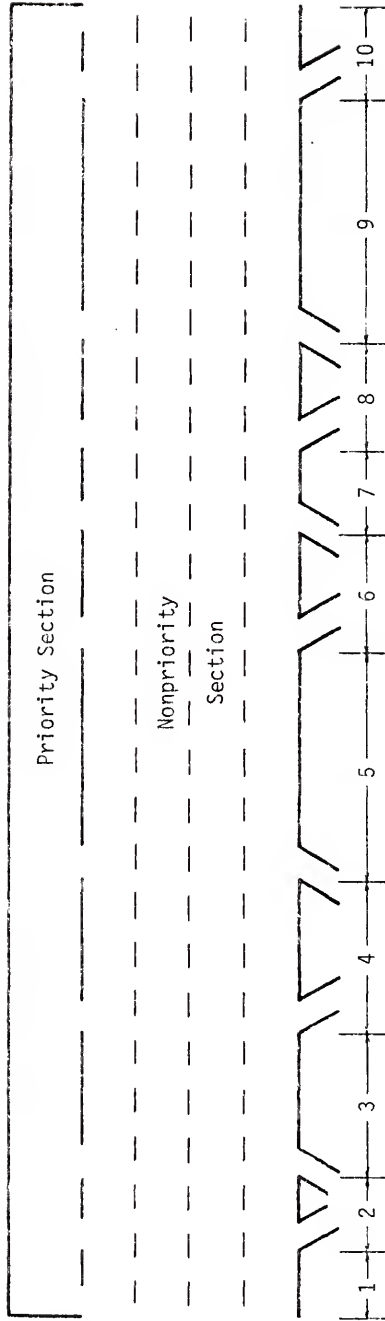


Figure 6.1 SCHEMATIC OF THE TEST SECTION FOR THE ENTRY/EXIT MODEL SENSITIVITY ANALYSES

this facility are 5 exit and 4 entrance ramps. For the purposes of this investigation, it will be assumed that the per lane capacities of the reserved and nonreserved sections are 1,500 and 2,000 vehicles per hour, respectively, the free flow speed for both sections is 60 mph, and that weaving delays are negligible. Additionally, the 10 freeway sections were defined to be between adjacent ramp noses or gores, and the candidate priority lane entry or exit points were located at the upstream end of each section and corresponded to the freeway entry or exit operation at that point. A summary of these operating conditions is presented in Table 6.1. The traffic flow patterns for this system were determined from field data and are presented in Table 6.2. The demand-speed relationships assumed for this system are shown in Figure 6.2.

An initial evaluation of this test system using the previously developed technique indicates that the minimum total travel time of 1,369 vehicle-hours is obtained with priority lane entry in section 1 and exit in section 10 for a car pool definition of 2 persons per vehicle. Using a car pool requirement of 3 ppv, this minimum total travel time increases to 1,709 vehicle-hours with entry points in sections 1, 3, 5, 7, and 9 and exits in sections 2, 4, 6, 8, 10.

An accepted traffic flow relationship is that for a roadway with constant demand, the unit travel time decreases as the capacity increases. This implies that the total travel time should also decrease with increasing capacity. In order to verify that this is reflected by the entry/exit model, a series of analyses were made with the assumed capacities varying.

Table 6.1 SUMMARY OF THE TEST SYSTEM OPERATING CHARACTERISTICS
FOR THE ENTRY/EXIT MODEL SENSITIVITY ANALYSES

Sec.	Length (feet)	Number of Lanes		Capacity (vph)		Entry/Exit	
		Priority	Nonpriority	Priority	Nonpriority	Priority	Nonpriority
1	500	1	4	1500	8000	Entry	Entry
2	448	1	4	1500	8000	Exit	Exit
3	2577	1	4	1500	8000	Entry	Entry
4	2075	1	4	1500	8000	Exit	Exit
5	3091	1	4	1500	8000	Entry	Entry
6	1644	1	4	1500	8000	Exit	Exit
7	1054	1	4	1500	8000	Entry	Entry
8	1506	1	4	1500	8000	Exit	Exit
9	3795	1	4	1500	8000	Entry	Entry
10	500	1	4	1500	8000	Exit	Exit

Table 6.2 TEST SYSTEM ORIGIN-DESTINATION CHARACTERISTICS

		Origin					Total
		1	2	3	4	5	
Destination	1	1.59					1.59
	2	3.31	0.04				3.35
	3	3.20	0.13	0.11			3.44
	4	4.33	0.30	0.30	0.03		4.96
	5	5.15	0.26	0.36	0.08	0.17	6.02
	6	61.35	3.01	9.08	3.93	3.27	80.64
Total		78.93	3.74	9.85	4.04	3.44	100.00

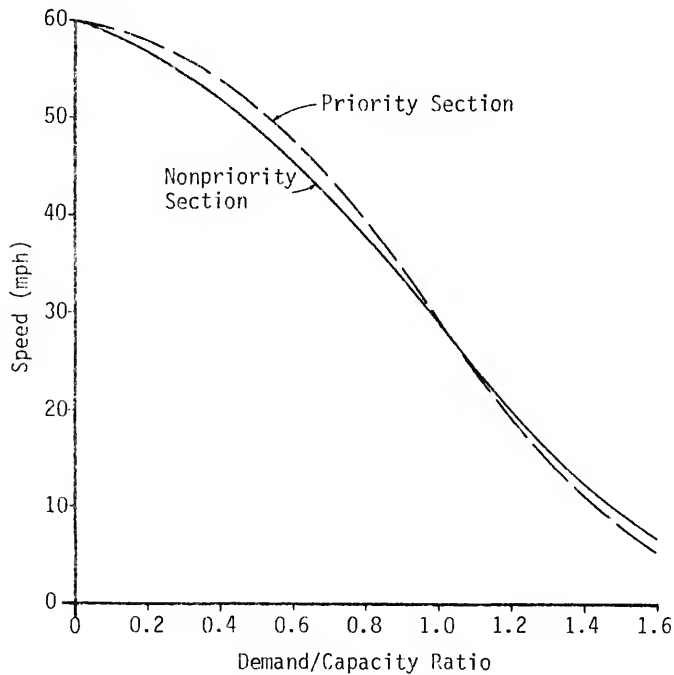


Figure 6.2 ASSUMED DEMAND-SPEED RELATIONSHIPS FOR THE ENTRY/EXIT MODEL SENSITIVITY ANALYSES

First, the priority lane capacity was varied from 70% to 130% of its original value, and the optimal entry/exit strategy was determined at each level. The results of these analyses, presented in Table 6.3, demonstrate that this expected relationship is maintained in this case. The weaker relationship found for the 3 ppv car pool requirement is a reflection of the fact that the system was over-restricted at that definition. The sensitivity of the model to variations in the priority capacity is shown in Figure 6.3.

Next, the nonpriority section capacities were varied in a similar fashion, and the optimal entry/exit configurations were determined for both car pool definitions. Again, the results of these analyses, Table 6.4, clearly reflect the anticipated relationship. As is shown in Figure 6.4, the total system travel time at optimality is a nonlinearly decreasing function of the capacity.

A second flow relationship which should be reflected by the model is that which exists between demand and travel time. As the total vehicular demand increases, the travel time on a roadway increases nonlinearly, provided other factors are constant. This implies that the total travel time within the system should also increase in a nonlinear fashion. Verification of the model with respect to this consideration was accomplished by allowing the total demand to vary from 70% to 130% of the original value and determining the optimal entry/exit strategy for each level. The results of these analyses are presented in Table 6.5 for car pool definitions of 2 and 3 persons per vehicle. The hypothesized nonlinearly increasing relationship

Table 6.3 MINIMUM TOTAL TRAVEL TIME FOR VARYING
PRIORITY SECTION CAPACITIES

Capa- city (vph)	Total Vehicle Hours	
	Car Pool = 2	Car Pool = 3
1050	1533.5	1713.3
1200	1470.9	1711.8
1350	1418.3	1710.2
1500	1369.0	1708.2
1650	1328.1	1707
1800	1287.9	1705.5
1950	1264.3	1704.6

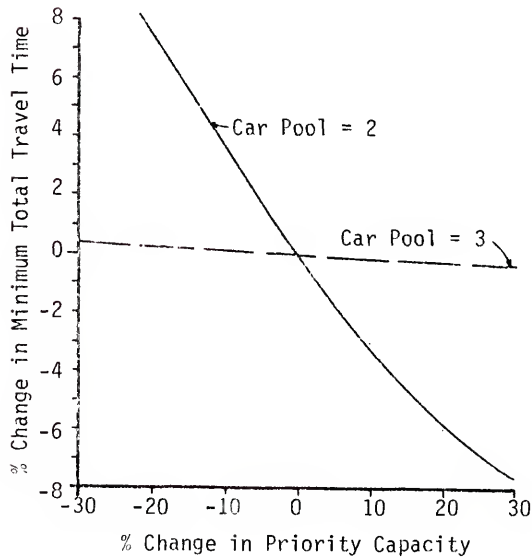


Figure 6.3 EFFECT OF VARYING PRIORITY SECTION CAPACITIES
ON OPTIMUM SYSTEM PERFORMANCE

Table 6.4 MINIMUM TOTAL TRAVEL TIME FOR VARYING
NONPRIORITY SECTION CAPACITIES

Capacity (vph)	Total Vehicle Hours	
	Car Pool = 2	Car pool = 3
5600	2697.1	4168.6
6400	2092.1	2856.9
7200	1688.2	2112.5
8000	1369.0	1708.2
8800	1249.3	1369.8
9600	1209.4	1237.6
10400	1169.5	1197.7

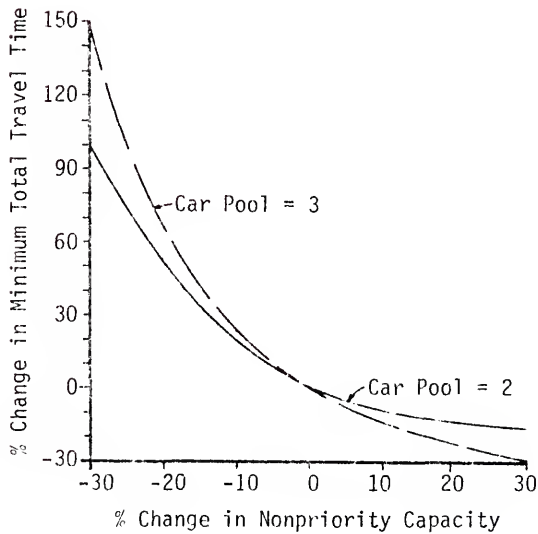


Figure 6.4 EFFECT OF VARYING NONPRIORITY SECTION CAPACITIES
ON OPTIMUM SYSTEM PERFORMANCE

Table 6.5 MINIMUM TOTAL TRAVEL TIME FOR VARYING LEVELS OF VEHICULAR DEMAND

Demand (veh)	Total Vehicle Hours	
	Car Pool = 2	Car Pool = 3
12482	766.2	799.6
14266	934.6	971.1
16049	1111.9	1275.4
17832	1369.0	1708.2
19615	1903.4	2284.3
21398	2502.7	3025.1
23182	3185.0	4199.4

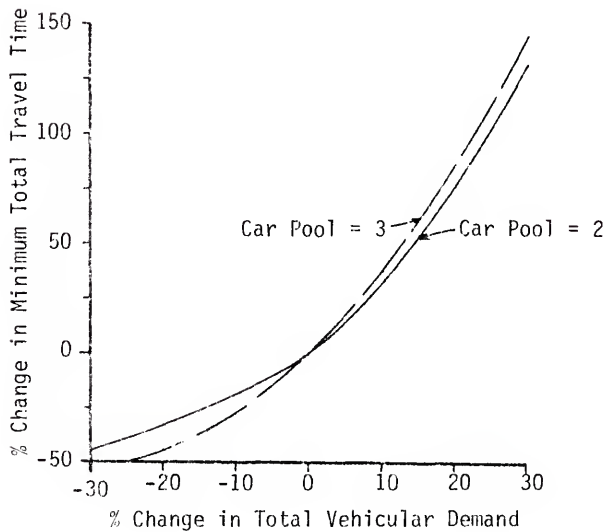


Figure 6.5 EFFECT OF VARYING LEVELS OF VEHICULAR DEMAND ON OPTIMUM SYSTEM PERFORMANCE

between total travel time and demand and the sensitivity of the model to demand variations are evident in the curves shown in Figure 6.5.

Postoptimality Analysis

A second check of the proposed model operation is a demonstration that the recommended entry/exit configuration is indeed an optimal strategy. This postoptimality analysis is intended to show that the model recommends an entry/exit strategy for a particular system such that total system travel time is minimized. Demonstration of this optimal state will require that 2 conditions be met:

1. The closure of all candidate entry/exit points not included in the recommended strategy does not increase the travel time.
2. The closure of any candidate entry/exit point that is included in the recommended strategy will increase the total travel time.

The test case selected for this evaluation was the basic test system described earlier, operating with a priority capacity of 1,200 vph and a car pool definition of 2 persons per vehicle. The recommended entry/exit configuration was priority entry in sections 1 and 3, and exit in section 10. With this strategy, the minimum total travel time was found to be 1,169 vehicle hours. The postoptimality analysis was then carried out in the following steps:

1. Evaluation of minimum total travel time with consideration of all nonrequired candidate entry/exit points disallowed

2. Evaluation of minimum total travel time with each required entry/exit point individually disallowed
3. Evaluation of minimum total travel time with the required entry/exit points jointly disallowed.

The results of these evaluations, presented in Table 6.6, show the following:

1. The total travel time was unaffected by closure of all nonrequired entry/exit locations.
2. The total travel time was increased by the closure of any single entry/exit point and also by the joint closure of these locations.

These results can then be used to infer that an optimal entry/exit configuration was selected.

Comparison with Simulation Technique

The final consideration to be investigated in conjunction with the validity of the proposed model is the overall accuracy of the total travel time predictions. For this evaluation, the travel times predicted by the entry/exit model, STRATEGY, will be compared to those predicted by a priority lane simulation model, PRIFRE [Minister et al., 1973], for a series of varying operating conditions.

The test system for this comparison must be developed in such a manner that the models will perform their analyses on a common basis, and that conflicts between the underlying model assumptions will be avoided. The primary difference in these models is that the entry/exit model determines the optimal entry/exit configurations for a given

Table 6.6 RESULTS OF POSTOPTIMALITY ANALYSIS
FOR THE ENTRY/EXIT MODEL

Action Taken	Resulting Minimum Total Travel Time
All candidate entry/exit locations considered	1169 vehicle hours
Closure of entry/exit points in sections 3,4,5,6,7,8, and 9	1169 vehicle hours
Closure of exit in section 10	1471 vehicle hours
Closure of entrance in section 2	1471 vehicle hours
Closure of entrance in section 1	1496 vehicle hours
Closure of exit in section 10 and entrance in section 2	1471 vehicle hours
Closure of exit in section 10 and entrances in sections 1 and 2	2052 vehicle hours

system; whereas, the PRIFRE model simulates the system operation under the assumption of an end-point strategy. This conflict can be eliminated by piecewise analyses and subsequent manual interfacing for the simulation process.

For this reason, a relatively simple test system is desirable. During the course of the sensitivity analyses, it was found that for a car pool definition of 2 persons per vehicle, the recommended entry/exit configuration was relatively stable for demands varying from 80% to 120% of the original value. This system was then adapted as a test case for these comparisons, with the only modification being an adjustment of the demands to represent hourly flows.

The models were then applied to the test case, and the results, shown in Table 6.7 and Figure 6.6, show that both models predicted similar operations in all cases. The discrepancies in these results averaged 3.7% with the maximum difference being 6.4%. Again, these differences are possibly due to variation in the treatment of the piecewise linear approximation of the demand-speed relationship between the two models.

Applications of the Model

Objective

In this section considerations will be directed toward the potential uses of the entry/exit strategy model. This presentation will not be a development of specific procedures to be followed in the application of the model; rather, it will serve to identify those situations in which the model can contribute to the investigation of an HOV priority lane treatment. It is quite likely that additional

Table 6.7 TOTAL TRAVEL TIMES PREDICTED BY
STRATEGY AND PRIFRE MODELS

De- mand (veh)	Total Vehicle Hours		% Deviation
	STRATEGY	PRIFRE	
14266	590	558	5.73
16049	700	691	1.30
17832	941	884	6.45
19615	1303	1262	3.25
21398	1683	1719	-2.09

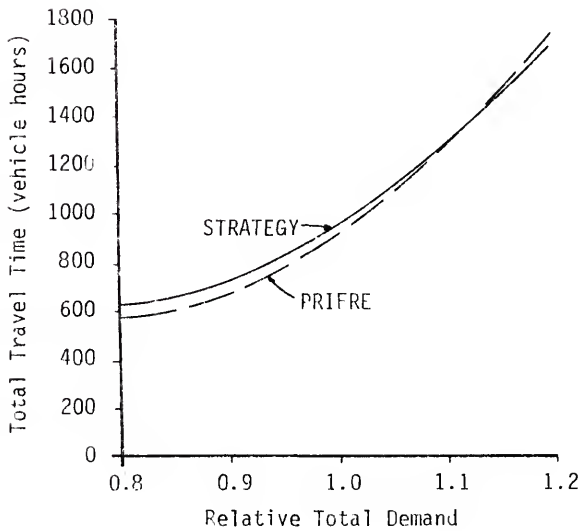


Figure 6.6 COMPARISON OF TOTAL TRAVEL TIME PREDICTIONS
OF STRATEGY AND PRIFRE MODELS

applications will be suggested by the characteristics of a particular situation, however, the discussion which follows addresses those areas of general interest.

Basic Priority Lane Entry/Exit Strategy

The most obvious application of this model is the formulation of a static priority lane entry/exit strategy. This strategy would be developed considering the demands, capacities, and operating characteristics for the full period of priority operation and would remain fixed throughout this time. Of the HOV priority lane systems implemented to date, all have adopted a fixed entry/exit strategy.

An application of the model for this purpose is relatively straightforward, as was previously discussed. However, 2 areas are sufficiently critical to warrant additional discussion at this point. The first of these is the identification of candidate entry and exit points. The structure of this model is such that priority lane interchange can only be considered at the upstream ends of the designated subsections. This assumption should not adversely affect the optimization process if the subsections are developed in such fashion as to minimize the possibility that entry or exit maneuvers would be required at other points. This can be achieved by defining each segment of the system in which the demand, capacity, and operating characteristics are constant as a separate subsection. This process will result in designating a candidate entry/exit point at each location where there is variation in the factors determining the system travel time.

Another important consideration is the possible need to operate portions of the system such that the demand exceeds the available capacity. In order to provide the information necessary to address this possibility, the demand-speed relationships would be extended into the oversaturated condition if the overall freeway system is operating at or near capacity. A convenient method of estimating these characteristics is the combination of the established speed-flow curves for undersaturated flow with estimated travel time relationships for the oversaturated condition. A variety of these estimation techniques have been developed for use in the transportation planning process and can be readily adapted for this purpose. Additional suggestions relating to this procedure are presented in Appendix A.

A useful extension of this application is the determination of the required priority system length. If the system initially considered is defined with sufficiently broad geographical boundaries, the optimization process can be used to determine the locations of the upstream and downstream ends of the priority lane. These terminal points are defined by the geographical limits of the recommended entry/exit strategy.

Temporal Variation of the Entry/Exit Strategy

Another consideration with respect to the priority lane entry/exit is the concept of a temporally varying strategy. Since the vehicular demand can be expected to vary in magnitude throughout the period of priority operation and flow patterns may shift during this same time, it is possible that a time-dependent entry/exit strategy will offer substantial improvements to the overall system operation. The

proposed entry/exit optimization model can be readily applied in an investigation of this concept.

For the previous application, the demands and flow patterns for the entire period of operation were considered. In developing a time-varying strategy, it is necessary to consider these factors for each of several consecutive time blocks comprising the total period. These blocks or "time slices" might normally be 15 to 30 minutes in length, depending on the particular situation. For each time slice, the procedures previously discussed would then be used to determine the optimal strategy for that period. The resulting strategies for the individual time slices can then be combined to define the optimal strategy as a function of time within the period of operation.

A logical extension of this application can be used to determine the time period during which the priority treatment should be provided. These temporal limits are identified in a manner similar to the determination of the geographical limits previously discussed. Simply stated, the period of operation can be defined as that period between the time when entry into the priority lane becomes feasible and the time when it ceases to offer travel time benefits. These are the times at which the optimal strategy does not require priority lane entry or exit. Through proper development of the demand-speed characteristics and the weaving delay factors, the model can be used to define these limits, as when a selected level of service is reached, or when the priority lane no longer offers some minimum travel time savings

to priority vehicles. Due to the general structure of the model, this secondary application would be appropriate only in those cases where the priority lane is used as a breakdown or refuge area outside the period of priority operation.

General Observations

Objective

In this final section, some general observations with respect to the priority lane entry/exit strategy will be presented. Inasmuch as the development of an optimal strategy is highly dependent on site-specific parameters, it is not practical to generalize the operation to the point where specific guidelines can be developed. However, during the course of investigating entry/exit strategies for a variety of hypothetical and "real-world" systems, some general trends have been noted. It is these observed trends that will be discussed in this section.

Overrestricted Facilities

The first point to be made is concerned with the entry/exit strategy for facilities with an overrestrictive car pool definition. If the number of occupants required for priority status is developed in a conservative manner or is based on providing a high degree of priority, it will be necessary to provide points of priority lane access or egress in most, if not all, freeway subsections with demand or capacity changes. This required strategy would approach the continuous entry/exit system previously described. The need for this is apparent, since to maintain the proper flow balance, the entry/exit

strategy must conform to the trip patterns of most, if not all, priority vehicles.

Underrestricted Facilities

In the case of priority systems with more liberal car pool definitions or lower levels of priority treatment, the optimal entry/exit strategy is more dependent on the flow balance between the 2 sections. This normally results in strategies similar to either the discrete or the end-point entry/exit system. In this case, it is not necessary or, in fact, may not be desirable to provide immediate priority lane access to all qualified vehicles. Thus, for these facilities, the entry/exit strategy may be more responsive to the more significant congestion and weaving problems.

Weaving Delays

The final observation to be made concerns the effect of weaving delays on the priority lane operation. It has been seen that in several cases where the priority section offers substantial travel time advantages over the nonpriority lanes, the delays encountered in weaving across the nonpriority section to enter the priority section or to exit from the freeway can negate or severely reduce the benefits of traveling in the reserved portion. Thus, in the development of an entry/exit strategy for a particular facility, it is important to adequately consider any existing or anticipated areas with significant weaving problems.

CHAPTER 7
A CASE STUDY:
THE I-95 PRIORITY LANE SYSTEM

Introduction

Objectives

In this chapter, the methodologies which have been developed for investigating certain operational and control elements of reserved high-occupancy vehicle (HOV) lanes on urban freeways will be applied to an existing priority lane system. The facility selected for this application is an HOV priority lane system now in operation on I-95 in Miami, Florida. In the analysis to follow, the physical and operational characteristics of the I-95 system will be identified, and the optimal car pool definition and entry/exit strategy will be investigated.

System Description

The I-95 HOV priority lane system is located in one of the major transportation corridors in Miami. This 10-mile corridor, which is shown in Figure 7.1, serves as a primary route for travel between the residential areas in northern Dade and southern Broward counties and the employment areas in the central and southern portions of Miami. In an attempt to alleviate the recurring rush-hour congestion on this freeway, the State of Florida, in cooperation with federal and local agencies, has embarked on a program to provide preferential treatment

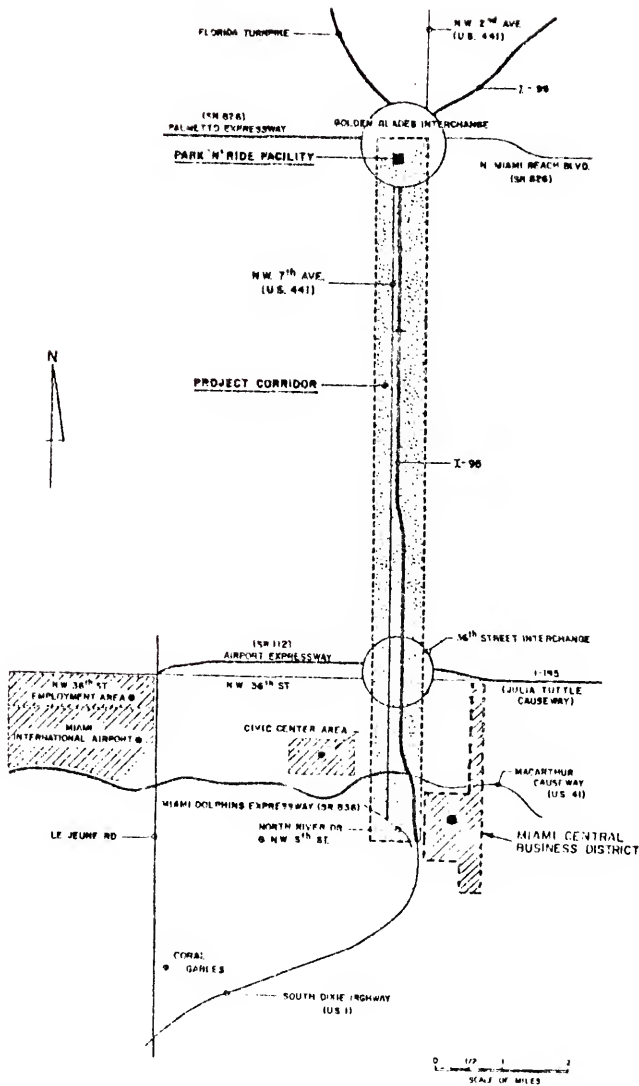


Figure 7.1 THE I-95 CORRIDOR

for high-occupancy vehicles using this facility [Florida Department of Transportation, 1972]. While the total program includes preferential treatments for buses on both I-95 and the parallel arterial, N.W. 7th Avenue, and HOV priority on I-95, this analysis will be limited to the I-95 system.

The overall geometric design of I-95 would be classified as relatively modern, although some individual ramp terminals do reflect substandard designs. This freeway is a 10-lane facility south of Airport Expressway Interchange, an 8-lane facility between the Airport Expressway and 135th Street Interchanges, and has 6 lanes north of 135th Street. A schematic of the freeway is presented in Figure 7.2. Prior to the implementation of the HOV lanes, a grass median separated the directional flows. This median was converted for use by priority vehicles through the construction of 2 paved lanes, immediately adjacent to and on the left of the original inside lanes, with a Jersey-type, concrete barrier separating the opposing movements. No physical separation of the normal and reserved lanes in the same direction was provided. The total length of the reserved lane implementation was slightly over 7 miles.

For the purposes of these analyses, the total length of freeway with priority lanes provided was subdivided into sections with both constant demand and capacity. Primarily, these sections are defined by adjacent ramp terminals, however, a lane drop between the Airport Expressway on-ramp and the 62nd Street off-ramp was also considered. This resulted in the formation of 18 analysis sections for the PM peak period. These sections range from 900 to 3,795 feet or 7.13 miles. In

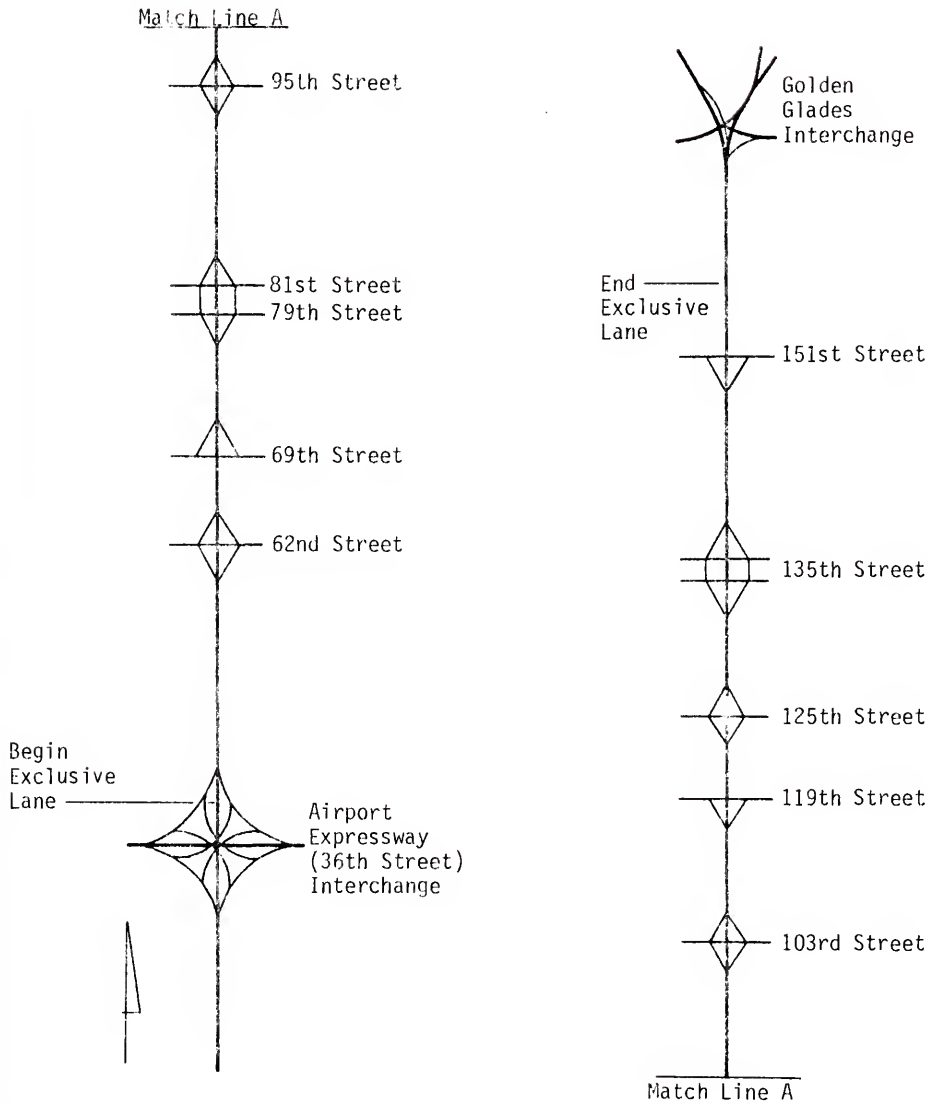


Figure 7.2 SCHEMATIC OF I-95

all sections, 1 lane was provided for priority use, while the number of nonreserved lanes varied from 3 to 6. The capacity of the reserved lane was estimated to be 1,500 vehicles per hour, since the overtaking and passing of slower-moving vehicles would not be possible in the single-lane operation. The capacities of the nonreserved lanes were based on the Highway Capacity Manual [Highway Research Board (HRB), 1965] figures for level of service C operation with a peak-hour factor of 0.91. A summary of the individual section characteristics is presented in Table 7.1.

The operational data required for these analyses of the I-95 system were developed from field studies carried out in conjunction with research into the traffic controls required for car pools and buses operating in priority lanes [Transportation Research Center, 1977]. Primarily, these data consist of system travel-times, volume counts, passenger occupancy studies, and origin-destination patterns for the freeway traffic.

First, the system travel-time and volume data were combined to develop estimates of the demand-flow relationships for this facility in the noncongested flow range. This curve was then extended into the congested flow realm with the technique outlined in Appendix A, and the resulting relationship was assumed to represent flow in the non-reserved lanes. For the flow in the priority lane, it was felt that the same demand-speed relationship would be valid, except for periods of low demand. This reasoning was based on the fact that in a single-lane flow situation, where overtaking and passing slower-moving vehicles is not possible, the maximum speed would be controlled by the

Table 7.1 I-95 ANALYSIS SECTIONS - PM PEAK PERIOD

Location	Entry/Exit		Section	Length (feet)	Number of Lanes		Capacity (vph)	
	EN	EX			Priority	Non-Priority	Priority	Non-Priority
Begin X-Lane	1		1	1390	1	3	1500	4350
Airport Xway On	2		2	1341	1	6	1500	9300
Lane Drop			3	3294	1	5	1500	7650
62 Off		1	4	900	1	4	1500	6000
62 On	3		5	1863	1	4	1500	6000
69 On	4		6	2577	1	4	1500	6000
79 Off		2	7	2075	1	4	1500	6000
81 On	5		8	3091	1	4	1500	6000
95 Off		3	9	1644	1	4	1500	6000
95 On	6		10	1054	1	4	1500	6000
103 Off		4	11	1506	1	4	1500	6000
103 On	7		12	3795	1	4	1500	6000
119 Off		5	13	1982	1	4	1500	6000
125 Off		6	14	1478	1	4	1500	6000
125 On	8		15	1880	1	4	1500	6000
135 Off		7	16	1890	1	3	1500	4350
135 On	9		17	3434	1	3	1500	4350
151 Off		8	18	2474	1	3	1500	4350
End X-Lane								

slower vehicles. It was then assumed that for levels of service A and B, the maximum priority lane speed would be constant at approximately 47 miles per hour. This speed was selected since it corresponded to minimum level of service B operation. These priority and nonpriority demand-speed relationships are shown in Figure 7.3.

Next, the available volume, occupancy and origin-destination data were used to determine both the vehicular and passenger demands for all freeway sections. Beginning with data from a series of point volume studies and the known freeway flow patterns, the vehicular demands were determined for each 30 minute interval between 3:30 and 6:30 PM, and for the entire peak period. The freeway origin-destination data are presented in Table 7.2, and the resulting section demands are presented in Table 7.3. Available data from vehicle occupancy studies, presented in Table 7.4, were then used to develop the passenger flow patterns for the individual freeway sections and for the entry and exit ramps. These freeway section demands, stratified by level of occupancy, were required for the determination of the optimal car pool size. The ramp demands were broken down into priority and nonpriority demands at all feasible car pool definitions, as required for the entry/exit analysis.

Analysis Procedure

After the development of these data, the proposed models were applied to the I-95 HOV priority lane system. First, the optimal car pool definition was determined for the individual freeway sections for each half-hour interval and the entire peak-period. For these analyses, it was estimated that the average violation rate would be 10% and the

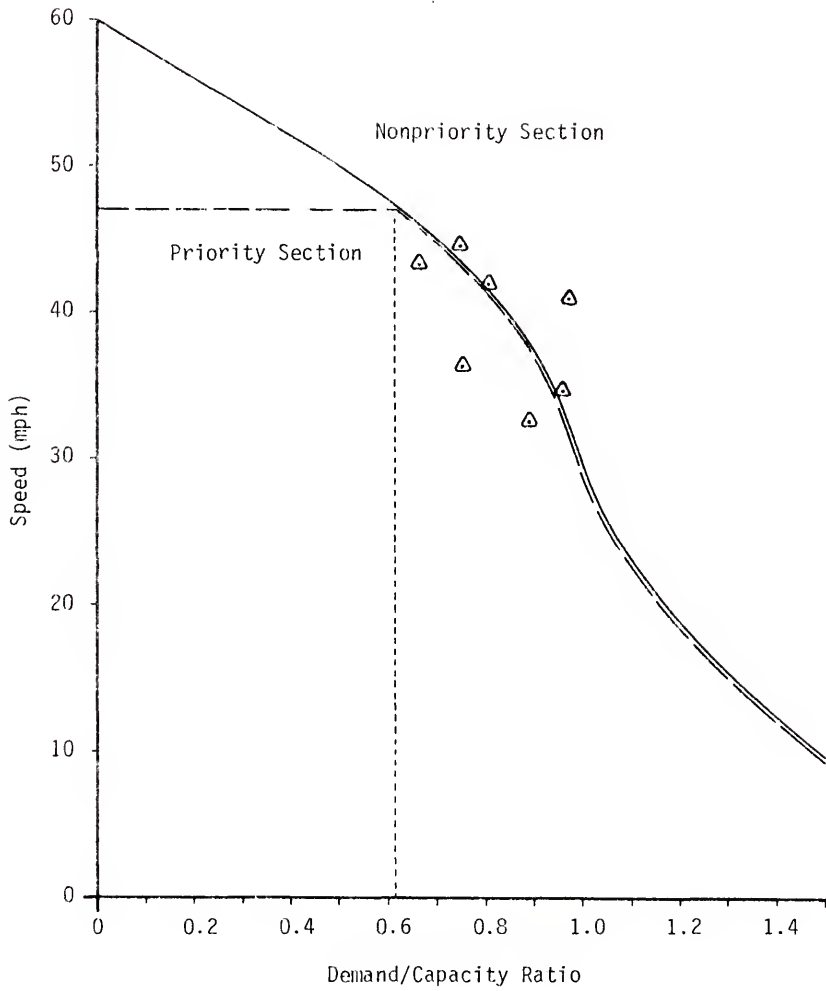


Figure 7.3 DEMAND-SPEED RELATIONSHIPS FOR THE I-95 SYSTEM

Table 7.2 I-95 ORIGIN-DESTINATION TABLE

	Exit									Total
	1	2	3	4	5	6	7	8	9	
1	0.32	1.53	1.62	1.98	1.99	2.10	3.64	0.84	18.91	33.43
2	0.89	1.51	1.23	1.68	2.46	2.00	3.13	0.79	19.84	33.53
3		0.18	0.09	0.31	0.28	0.40	0.91	0.15	3.56	5.88
4		0.04	0.12	0.27	0.24	0.25	0.25	0.06	2.19	3.42
5			0.10	0.27	0.33	0.37	0.97	0.24	6.76	9.04
6				0.03	0.07	0.28	0.44	0.17	2.72	3.71
7					0.16	0.24	0.35	0.16	2.24	3.15
8							0.21	0.09	3.45	3.75
9								0.11	3.98	4.09
Total	1.71	3.26	3.16	4.54	5.53	5.64	9.90	2.61	63.65	100.00

E n t r a n c e

Table 7.3 I-95 ANALYSIS SECTION DEMANDS

S E C.	Volume During Period						
	3:30 4:00	4:00 4:30	4:30 5:00	5:00 5:30	5:30 6:00	6:00 6:30	3:30-6:30
1	1501	1944	1929	1777	1515	1328	9,994
2	3006	3895	3864	3559	3035	2659	20,018
3	3006	3895	3864	3559	3035	2659	20,018
4	2929	3795	3765	3468	2957	2591	19,505
5	3193	4137	4104	3781	3224	2825	21,264
6	3347	4336	4301	3963	3379	2961	22,287
7	3200	4146	4113	3789	3231	2831	21,311
8	3606	4672	4635	4270	3641	3190	24,013
9	3464	4488	4452	4102	3498	3065	23,069
10	3631	4704	4667	4299	3666	3212	24,179
11	3427	4440	4404	4053	3460	3032	22,821
12	3568	4623	4586	4225	3603	3157	23,762
13	3320	4302	4267	3931	3352	2937	22,110
14	3067	3974	3942	3632	3097	2713	20,424
15	3235	4192	4158	3831	3267	2862	21,545
16	2791	3616	3587	3305	2818	2469	18,585
17	2974	3854	3823	3522	3003	2631	19,807
18	2857	3702	3672	3383	2885	2528	19,027

Table 7.4 PASSENGER OCCUPANCY DISTRIBUTIONS
FOR I-95 SYSTEM

			Analysis Period						
			3:30 to 4:00	4:00 to 4:30	4:30 to 5:00	5:00 to 5:30	5:30 to 6:00	6:00 to 6:30	Total Peak Period
Reserved Section	Average Occupancy		1.465	1.881	2.011	1.930	1.854	1.381	1.752
	Number of Occupants (% of All Vehicles)	1	65.3	30.6	24.1	23.4	31.3	68.2	40.2
		2	29.6	55.8	58.2	63.8	58.3	27.1	48.8
		3	4.3	9.2	11.4	9.8	5.7	3.3	7.3
		4	2.1	3.1	5.0	2.5	3.3	1.3	2.9
		5	0.5	0.8	1.3	0.5	1.5	0.1	0.8
Nonreserved Section	Average Occupancy		1.419	1.431	1.320	1.320	1.406	1.365	1.371
	Number of Occupants (% of All Vehicles)	1	69.2	68.3	77.1	74.4	70.5	72.4	72.3
		2	23.9	24.2	17.5	20.9	22.3	22.7	21.7
		3	3.9	4.4	2.7	3.1	4.0	2.6	3.4
		4	1.8	2.5	1.8	1.3	2.5	1.6	1.9
		5	1.2	0.7	0.9	0.2	0.7	0.7	0.7
Total System	Average Occupancy		1.427	1.438	1.406	1.421	1.475	1.361	1.429
	Number of Occupants (% of All Vehicles)	1	68.1	65.2	70.6	66.0	64.4	71.8	67.5
		2	24.9	29.9	22.5	28.0	27.8	23.3	25.7
		3	4.0	1.4	3.8	4.2	4.3	2.7	4.0
		4	1.9	2.8	2.2	1.5	2.7	1.5	2.1
		5	1.1	0.7	0.9	0.3	0.8	0.7	0.7

average nonutilization rate would be 20%. The 10% violation rate reflects the experience of other preferential treatment systems, and the nonutilization rate was estimated on the basis of preliminary studies on I-95. Additionally, all analyses were carried out for minimum vehicle-hours of travel in addition to minimum passenger-hours. Next, the optimal entry/exit strategy was determined for each 30 minute interval, as well as for the 3-hour peak period. In these analyses, it was assumed that weaving delays were negligible.

Car Pool Definition Analysis

General Comments

Prior to a discussion of the car pool definition analyses results, some general comments are appropriate. First, for all analyses, the fixed level of priority constraint was adopted to determine the minimum amount of HOV priority to be given. This constraint, with an assumed priority differential of 0.05, would insure only a minimal level of priority and would thereby result in a liberal car pool definition. This relatively low minimum level of priority should be kept in mind when considering the results of these analyses. Second, one should be aware that the objective of preferential treatment systems is not solely the reduction of congestion. It is also the provision of incentives for motorists to travel in high-occupancy vehicles, which increases the passenger-carrying capability of the roadway and reduces the total energy requirements of the travel. With these goals, it is imperative that the priority lane operate at a higher level of service than the adjacent nonpriority lanes, or this incentive will not be provided.

Basic Car Pool Definition

In order to develop a single car pool definition for all sections during the entire period of operation, an evaluation of the results of the peak-period analyses is appropriate. These analyses, summarized in Appendix B, Table B.1, readily demonstrate that the minimum level of occupancy which should be required for priority status is either 2 or 3 persons per vehicle. A close examination of these results reveals that the optimal car pool definition lies between these 2 levels in all but 2 analysis sections, as is shown in Figure 7.4.

In order to make the choice between the 2 and 3 ppv car pool definition, other factors must be considered. The first of these is the estimated hours of travel for each of the candidate definitions. For the 2 ppv definition, it was estimated that 7,918 passenger-hours and 5,548 vehicle-hours of travel would be required, whereas, for a requirement of 3 ppv, 9,073 passenger-hours and 6,607 vehicle-hours would be expended. At optimality, the total travel time was estimated to be 7,760 passenger-hours and 5,557 vehicle-hours.

On the surface, these figures seem to indicate that the 2 ppv definition would be the preferable alternative. However, another factor should be considered. This is the degree of priority given to the high-occupancy vehicles under each definition. The degree of priority is defined as the ratio of the nonreserved section demand/capacity ratio, and should be greater than 1.0 if preferential treatment is to be realized. An examination of this parameter as shown in Figure 7.5 indicates that the 3 ppv definition provides an acceptable degree of priority in all but 1 section, while the 2 ppv definition fails to

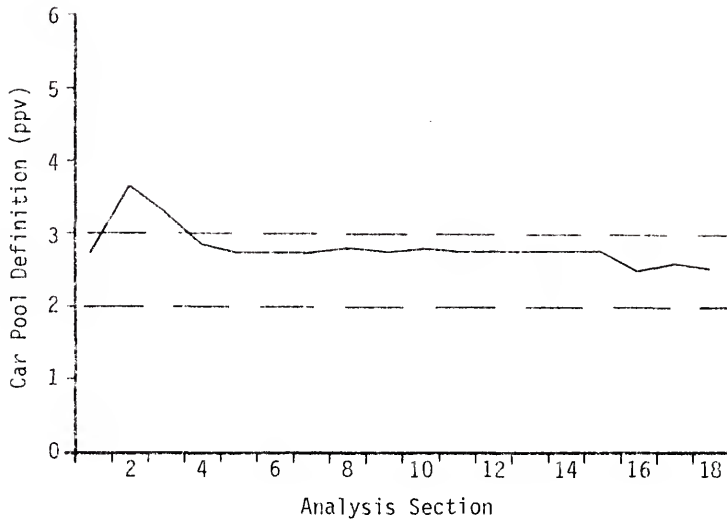


Figure 7.4 OPTIMUM CAR POOL DEFINITIONS FOR MINIMUM PASSENGER HOURS DURING THE PERIOD 3:30 TO 6:30 PM

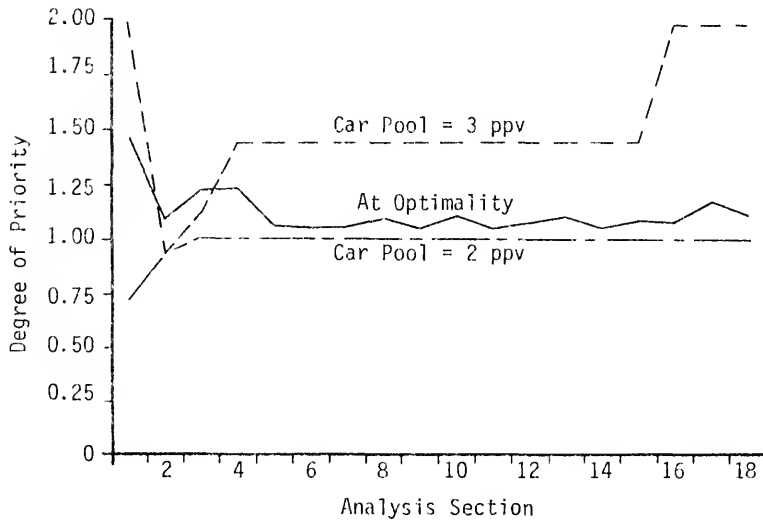


Figure 7.5 DEGREE OF PRIORITY FOR MINIMUM PASSENGER HOURS DURING THE PERIOD 3:30 TO 6:30 PM

provide a degree of priority greater than 1.0 in any section. In fact, it was only under the assumption of user optimization (equalizing the sections' demand/capacity ratios) that a degree of priority of the level 1.0 was attainable. Thus, it is now apparent that while a car pool definition of 2 ppv would require lower total travel times, it would not allow the provision of any preferential treatment for high-occupancy vehicles.

On the basis of these findings, a minimum level of passenger occupancy of 3 persons per vehicle is preferred as the basic car pool definition. This definition will provide a degree of treatment for high-occupancy vehicles, although a somewhat higher time-penalty would be paid by the nonpriority traffic than for the lower definition.

Temporal Variation

Having investigated the basic car pool definition for this system, consideration can now be given to the potential benefits of a time-varying definition. Appropriate for this investigation is an evaluation of the results of the system analyses by half-hour time intervals, summarized in Appendix B, Tables B.2 through B.7.

These results show that, in the majority of the cases, the choice is once again between a 2 ppv or 3 ppv minimum car pool requirement. Some isolated sections do fall outside this choice during selected intervals, particularly near the beginning and end of the peak period. However, for 14 out of the 18 total sections, the optimal car pool definition was found to be between 2 and 3 ppv for all intervals. For these 14 sections, the 3 ppv definition would again seem to be the best choice since the maximum degree of priority allowed by a

2 ppv definition would be 1.0. However, as in the previous case, the 2 ppv definition would result in lower vehicle-hours and passenger-hours of travel.

In light of the overall stability of this system, it is doubtful that the benefits which could be realized through temporal variation of the car pool definition would be of sufficient magnitude to justify the additional expense of providing this type operation.

Spatial Variation

A final strategy which can be considered is that of a spatially varying car pool definition. Under this type of operation, each section would have an independent car pool requirement, based only on the flows within that particular section. This approach can be considered with either the basic or the time varying car pool definition. In the former case, the resulting operation would be a simple, spatially varying definition, while in the latter case, the operation would approximate a real-time system.

Considering the potential of spatial variation in conjunction with the basic car pool definition, a review of the information in Figures 7.4 and 7.5, and in Table B.1 is required. From this data, it is apparent that for minimum passenger-hours of travel, spatial variation of the car pool definition would be appropriate only in sections 2 and 3. For section 2, this approach would result in a degree of priority greater than 1.0 if a definition of 4 ppv were used. However, this would also increase the total passenger hours of travel by 1.3% over the 3 ppv definition. In section 3, adopting a definition

of 4 ppv would result in an increase in total passenger hours of 1.5% and an increase in the degree of priority from 1.13 to 1.44 when compared to the 3 ppv definition.

Examination of the analysis results in Tables B.6 through B.7, shows that in only 12 of the 108 section time-periods a car pool definition other than 3 ppv would be required. Those instances in which the definition might vary are primarily confined to the first 3 sections, and within this block, they tend to be somewhat erratic. On the basis of these observations, it is not likely that the added cost of this operation would be justified by the potential benefits.

In summary, it does not appear that the use of a spatially varying car pool definition, either with or without temporal variation, would be appropriate for the I-95 system.

Priority Lane Entry/Exit Analysis

General Comments

The analysis of the priority lane entry/exit strategy for the I-95 system was approached in much the same manner as the investigation of the optimal car pool definition. That is, the system operation was analyzed by half-hour time intervals, as well as for the entire peak period. Within each analysis block, basic car pool definitions of the 2, 3 and 4 ppv were considered independently. As a result of the findings of the car pool definition analysis, however, the analyses for a 4 ppv car pool requirement will not be considered in subsequent discussions. Additionally, temporal or spatial variations in the car

pool definition were not considered, inasmuch as these alternatives were not found to be feasible for the I-95 system.

The results of these analyses for the 2 and 3 ppv car pool definitions are presented in Tables 7.5 and 7.6, respectively.

Basic Strategy

An examination of the analyses results presented in Table 7.5 reveals that for the entire peak period, an entry/exit strategy with points of access and egress in sections 1, 2, 3, 5, 6, 7, 8, 13, 14, 15, 17, and 18, as well as at the downstream end of the priority lane is the preferred configuration for a minimum car pool requirement of 2 persons per vehicle. For the 3 ppv car pool definition, priority lane entry or exit should be provided in all sections with the exception of section 2, as shown in Table 7.6. Under either basic car pool definition, these strategies will result in optimum system performance, as measured by the total vehicle hours of travel during the peak period.

Temporal Variation

With regard to an entry/exit strategy that varies with the peak period, an examination of Table 7.5 shows that this approach appears to be feasible for the lower car pool definition of 2 ppv. This time varying strategy would involve entry to the priority lane in section 3 and exit from the lane at its downstream end at all times, and no entry or exit in sections 4 and 16 at any time. The remaining sections would then have entry or exit provisions at various times within the peak period. For the car pool requirement of 3 ppv, however,

Table 7.5 OPTIMAL PRIORITY LANE ENTRY/EXIT STRATEGY FOR MINIMUM CAR POOL REQUIREMENT OF 2 PERSONS PER VEHICLE

Section		Candidate Entry/Exit Strategy	Optimal Entry/Exit Strategy					
No.	Description		Total Peak Period	3:30 to 4:00	4:00 to 4:30	4:30 to 5:00	5:00 to 5:30	5:30 to 6:00
1	Begin Exclusive Lane	Entry						6:00 to 6:30
2	Airport Expressway On	Entry						
3	Lane Drop	Entry						
4	62nd Off	Exit						
5	62nd On	Entry						
6	69th On	Entry						
7	79th Off	Exit						
8	81st On	Entry						
9	95th Off	Exit						
10	95th On	Entry						
11	103rd Off	Exit						
12	103rd On	Entry						
13	119th Off	Exit						
14	125th Off	Exit						
15	125th On	Entry						
16	135th Off	Exit						
17	135th On	Entry						
18	151st Off	Exit						
	End Exclusive Lane	Exit						

Note: Shading indicates sections with no entry or exit provisions.

Table 7.6 OPTIMAL PRIORITY LANE ENTRY/EXIT STRATEGY FOR MINIMUM CAR POOL REQUIREMENT OF 3 PERSONS PER VEHICLE

Section		Candidate Entry/Exit Strategy	Optimal Entry/Exit Strategy					
No.	Description		Total Peak Period	3:30 to 4:00	4:00 to 4:30	4:30 to 5:00	5:00 to 5:30	5:30 to 6:00
1	Begin Exclusive Lane	Entry						6:00 to 6:30
2	Airport Expressway On	Entry						
3	Lane Drop	Entry						
4	62nd Off	Exit						
5	62nd On	Entry						
6	69th On	Entry						
7	79th Off	Exit						
8	81st On	Entry						
9	95th Off	Exit						
10	95th On	Entry						
11	103rd Off	Exit						
12	103rd On	Entry						
13	119th Off	Exit						
14	125th Off	Exit						
15	125th On	Entry						
16	135th Off	Exit						
17	135th On	Entry						
18	151st On	Exit						
	End Exclusive Lane	Exit						

Note: Shading indicates sections with no entry or exit provisions.

Table 7.6 clearly shows that little potential for a time-variant strategy exists. For this definition, a fixed, continuous entry/exit strategy, with the exception of section number 2, would be most appropriate.

Summary of Findings

The preceding analyses have demonstrated the following with respect to the operation of the I-95 priority lane system during the PM peak period.

1. Based on the stated assumptions, a minimum car pool requirement of 2 persons per vehicle would result in both minimum vehicle-hours and minimum passenger-hours of travel.
2. The 2 ppv car pool definition would fail to provide the desired preferential treatment for high-occupancy vehicles as would the 3 ppv definition, for the stated assumptions.
3. The use of a temporally or spatially varying car pool definition would not produce substantial improvements in the system operation.
4. For a minimum car pool requirement of 2 persons per vehicle, a discrete entry/exit strategy would be appropriate, with potential for implementation of a time-varying configuration.
5. Under a car pool definition of 3 persons per vehicle, a continuous entry/exit strategy would be required and temporal variation of this strategy would not be beneficial.

In consideration of these findings and the underlying motivations for providing preferential treatment for high-occupancy vehicles, it is suggested that the I-95 priority lane system operate during the PM peak period with a minimum car pool requirement of 3 persons per vehicle and a continuous entry/exit strategy. After a suitable period of operation, the assumed violation and nonutilization rates should be evaluated and, if necessary, additional system analyses be carried out.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

This dissertation has proposed techniques for determining optimal control parameters and operating strategies for high-occupancy vehicle (HOV) priority lanes on urban freeways. In addition to the development of these techniques, the validity of the optimization models was demonstrated, and a case study application was presented. The following conclusions and recommendations are offered as a result of this study.

Conclusions

With respect to the car pool definition model, it is concluded that the proposed methodology represents a viable, useful technique for investigating HOV priority lane systems. This is supported by the following considerations:

1. The model is based on a proven technique, linear programming, which has been shown to be applicable in traffic flow analyses.
2. The fully developed model accurately reflects the operation of a priority lane system. This was demonstrated by both the sensitivity analyses and the comparison with a proven simulation model.

3. The proposed technique can be used to address a variety of considerations with respect to HOV priority lane systems.
4. The case study application demonstrated that useful results are obtained from investigations of "real-world" systems.

These investigations also support the following observations regarding the operation of a priority lane system:

1. Violation of the minimum occupancy restrictions by nonqualified vehicles has an adverse effect on the minimum passenger-hours of travel, but does not affect the minimum vehicle-hours. Similar results were found for the nonutilization of the priority lane by high-occupancy vehicles.
2. The proposed level of priority constraints are an effective means by which the degree of preferential treatment given to priority vehicles can be controlled.

It is further concluded that the proposed priority lane entry/exit model is also an accurate and useful evaluation technique. This conclusion is supported by the following observations:

1. The model is based on a proven system optimization technique, network flow analysis.
2. As fully developed, the model accurately predicts the operation of the priority lane system. Again,

this was shown with a sensitivity analysis and a comparison with an accepted simulation model.

3. The entry/exit model can be used in investigations of several aspects of priority lane operations.
4. The case study application demonstrated that meaningful results are obtained for "real-world" systems.

Additionally, this work also supports the following observations with respect to priority lane entry/exit strategies:

1. For overrestricted facilities, i.e., those for which the minimum occupancy requirement is higher than necessary, a continuous or nearly continuous entry/exit strategy is appropriate. In this case, the optimal strategy is primarily controlled by the origin-destination patterns of the priority vehicles.
2. For underrestricted facilities, i.e., those for which the minimum occupancy requirement is lower than the optimal value, a discrete or end point strategy is preferable. The optimal strategy in this case is more sensitive to weaving delays and the development of congestion.

Although the computational requirements of an application of either of the proposed models were not directly addressed, it is apparent from the basic structure of the models and the analyses

presented in Chapter 7 that these can be substantial. In most cases, manual computations would not be practical. This results from the following:

1. The car pool model requires an iterative application of a linear programming optimization model. Even recognizing that only 3 to 4 iterations would be necessary for most situations, the linear programming solution for each step requires a significant computational effort.
2. An application of the car pool model can also require a number of independent analyses, each necessitating the computational efforts described above.
3. While the entry/exit model is not an iterative process, the application of the network flow analysis technique to a system with a realistic number of nodes and flow arcs requires a substantial computational effort for any solution methodology.
4. An investigation of the optimal entry/exit strategy for a "real-world" system, can itself require several applications of the proposed model.

As a result of these computational difficulties, it is suggested that the computerized models, CARPOOL and STRATEGY, are quite useful tools for applications of the proposed methodologies. These programs are

based directly on the algorithms presented in the development of the models and greatly reduce the computational requirements of an application of either technique. This computational assistance will undoubtedly promote future utilization of these proposed methodologies.

Recommendations

Perhaps the most important recommendation for future research which was identified in this work is the need to measure the effect of the degree of preferential treatment which is provided for high-occupancy vehicles. This factor is critical to the success or failure of an HOV priority lane project. Specific questions which merit investigation in this area include the following:

1. At what degree of priority can a modal shift into high-occupancy vehicles be expected to begin?
2. At what degree of priority does the shift to high-occupancy vehicles cease?
3. What is the relationship between changes in degree of priority and HOV travel within the effective degree of priority range?
4. How do motorists perceive the degree of priority?

Another area in which additional research is needed is that of priority lane violation and nonutilization. The factors which control these parameters should be identified and, to the extent possible,

quantified. Additionally, the possibility of a relationship between violation and nonutilization should be considered, and if one does exist, it should be identified.

A third relationship which should be investigated is the effect of nonpriority section speed on priority lane speed. Is it realistic to assume that the priority flow speeds are independent of nonpriority speeds, or is there some maximum speed differential which can be maintained?

In the car pool definition process it was assumed that under certain conditions user optimization can be considered. This user optimization is, in essence, the self-imposed balance of the demand-to-capacity ratios in the priority and nonpriority sections for under-restricted systems. The question which should be addressed, preferably with field experimentation, is whether this is a realistic assumption. If it is not, research should determine how users will balance priority lane operation.

As a final recommendation pertaining to this work, consideration should be given to the treatment of microscopic weaving delays in the car pool definition process. It is conceivable that these delays could have an effect on the recommended minimum car pool size. If it is found that weaving delays are a significant factor in the selection of a car pool definition, the optimization model structure should be adjusted accordingly.

Within the general area of HOV priority lane systems, some additional investigations are warranted. These include establishing physical design guidelines where the priority lane design should vary

from current freeway design standards, and the development of operational parameters for these systems. These operational considerations should include the speed-flow relationships, the per-lane vehicular capacities, and the merging characteristics for priority lane access and egress.

APPENDIX A

DEVELOPMENT OF DEMAND-SPEED RELATIONSHIPS

One of the more important components required for application of the models proposed in this work is the demand-speed relationship for the facility under investigation. The importance of this relationship is derived from the selection of total travel time as the objective criterion for both optimization techniques. Within these methodologies, the unit travel time, or trip time, is a primary factor in the cost coefficients for this objective function.

For noncongested flow conditions, the determination of this relationship is a reasonably straightforward task. A series of field volume and travel time studies can be conducted, and the results used to determine the speed-flow curve for the particular facility. In this region, the speed-flow relationship is equivalent to the demand-speed relationship. Alternatively, a theoretical speed-flow curve for traffic flow could be assumed for the facility under investigation. Examples of these curves can be found in any of several transportation engineering references, such as the Transportation and Traffic Engineering Handbook [Baerwald, 1976] or the Highway Capacity Manual [HRB, 1965]. Typical of these curves for a volume-travel time relationship is the one shown in Figure A.1. This figure also presents a typical total travel time curve.

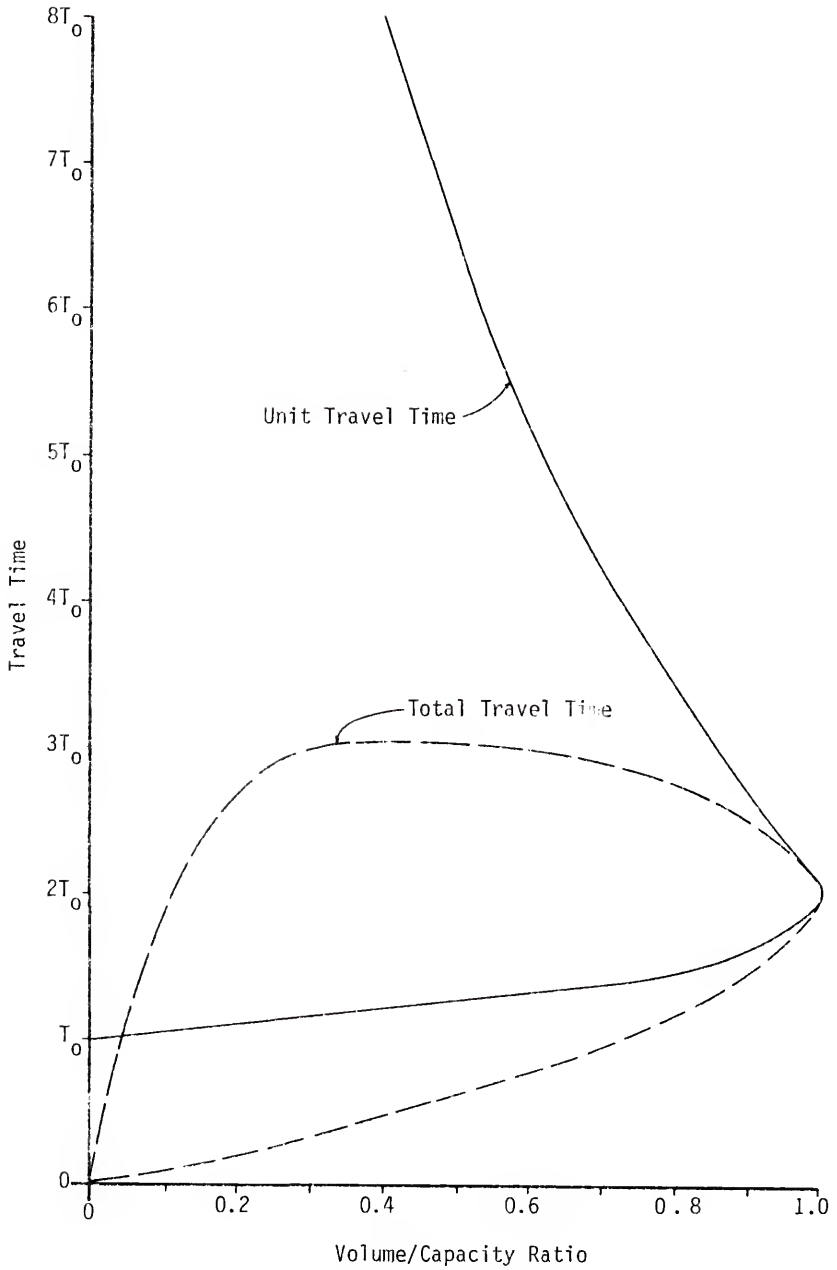


Figure A.1 TYPICAL VOLUME - TRAVEL TIME CURVES

In these curves, the lower limb of each represents flow in the noncongested region. In this area, the system demand is equal to the volume, so these lower limbs also represent typical demand-travel time curves. However, as the demand exceeds the system capacity, the volume is no longer equal to the demand, and, as the congestion increases, the unit travel time continues to increase, while the volume or flow rate decreases. This results in the relationships represented by the upper limbs of these curves.

Thus, this approach can only be used to develop demand-speed relationships for noncongested flow. In order to extend this relationship into the congested flow region, field data might also be used. However, accurate field determination of system demand in this condition is, at best, a difficult task. As an alternative to the development of the demand-speed relationship from field data, one of a number of travel time estimation techniques may be employed.

A survey of travel time forecasting techniques was included in a study by Huber et al. [1965], in which several of these techniques were identified. These techniques, developed primarily for use in the transportation planning process, included the following:

1. Smock Technique. This is an iterative assignment technique in which the travel times are estimated according to the following equation:

$$T_a = T_0 \cdot e^{(v/c) - 1}, \quad T_a \leq 5T_0 \quad (A.1)$$

Where: T_a = estimated travel time

T_0 = travel time at a v/c ratio of 1.0.

2. Bureau of Public Roads (BPR) Technique. This is also an iterative assignment in which the unit travel times are estimated according to Equation A.2.

$$T_a = T_0 \cdot [1.0 + 0.15(v/c)^4] \quad (A.2)$$

Where: T_a = estimated travel time

T_0 = travel time at zero volume.

3. Schneider Technique. This is a noniterative assignment technique in which the link travel times are estimated with the following equation:

$$T_a = T_0 \cdot (2)^{v/c}, \quad T_a \leq 4T_0 \quad (A.3)$$

Where: T_a = estimated travel time

T_0 = travel time at zero volume.

As a result of the comparisons made with these techniques, it was concluded that none of the 3 techniques was clearly superior to the others.

For the purposes of illustrating the extension of the demand-speed relationships into the range of congested flow and for use in applications of the proposed models, a modified BPR technique was selected. The original BPR function shown in Figure A.2 was felt to produce excessive deviation from the noncongested flow curve in the flow range near capacity, so Equation A.2 was modified to the general form of Equation A.4.

$$T_a = T_0 \cdot [1 + (D/C)^n] \quad (A.4)$$

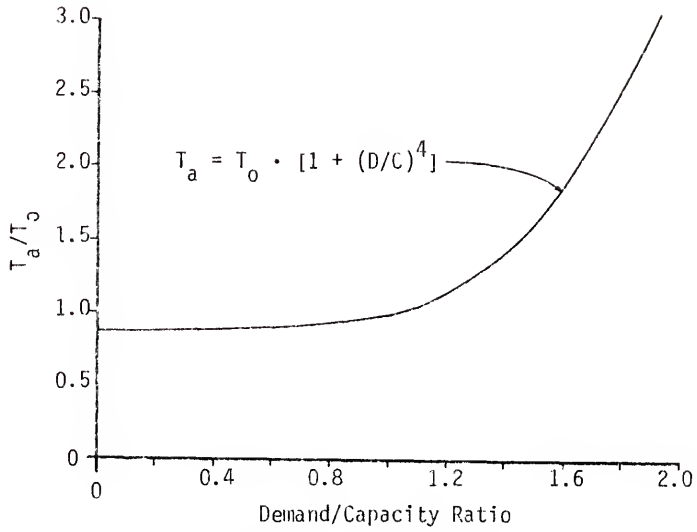


Figure A.2 ORIGINAL BUREAU OF PUBLIC ROADS FUNCTION

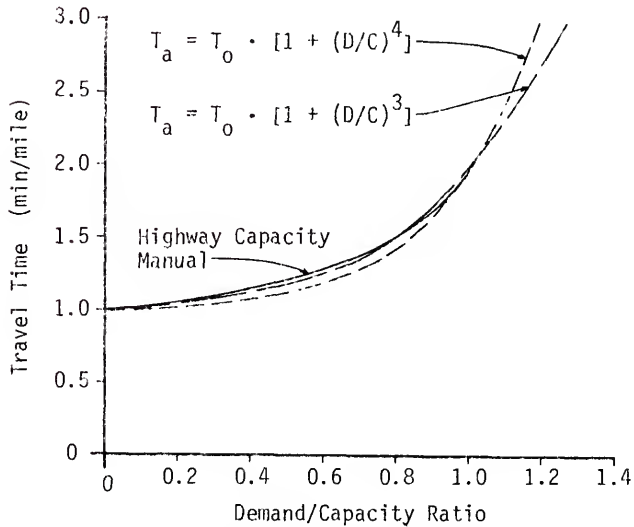


Figure A.3 COMPARISON OF MODIFIED BUREAU OF PUBLIC ROADS FUNCTION AND HIGHWAY CAPACITY MANUAL CURVE

Where: T_a = estimated travel time
 T_0 = travel time at capacity
 n = exponential constant.

Subsequent evaluation of different exponential constants resulted in satisfactory approximations for values of $n=3$ and $n=4$. Figure A.3 shows the modified BPR functions for these values in relation to the noncongested flow curve developed from the Highway Capacity Manual (HCM) speed-flow curves. These values were then correlated to the full known speed-flow curves, as summarized in Table A.1, and an exponential constant of 4 was selected for subsequent use.

At this point, there still remain two demand-travel time curves which must be combined for use in applications of the proposed models. Inasmuch as these models utilize a piecewise linear approximation of the demand-speed relationship, a graphical solution for the composite curve will be adequate. Such a graphical solution is shown in Figure A.4, and its corresponding demand-speed curve is presented in Figure A.5.

Table A.1 CORRELATION OF BUREAU OF PUBLIC ROADS FUNCTION AND HIGHWAY CAPACITY MANUAL CURVE

D/C Ratio	Travel Time (min/mile)				Speed (mph)				Flow Rate (vph/unit capacity)			
	HCM Curve		BPR Function		HCM Curve ($k < k_j$)	BPR Function		HCM Function	HCM Curve		BPR Function	
	$k < k_j$	$k > k_j$	Exp=3	Exp=4		Exp=3	Exp=4		$k < k_j$	$k > k_j$	Exp=3	Exp=4
0	1.000	∞	1.000	1.000	60.0	60.0	60.0	60.0	0	0	0	0
0.1	1.034	30.000	1.001	1.001	58.0	59.9	60.0	60.0	0.1	0.1	0.003	0.001
0.2	1.071	17.143	1.008	1.002	56.0	59.5	59.9	59.9	0.2	0.2	0.023	0.005
0.3	1.111	10.000	1.027	1.008	54.0	58.4	59.5	59.5	0.3	0.3	0.079	0.024
0.4	1.154	8.000	1.064	1.025	52.0	56.4	58.5	58.5	0.4	0.4	0.181	0.075
0.5	1.223	6.667	1.125	1.063	49.0	53.3	56.5	56.5	0.5	0.5	0.333	0.177
0.6	1.277	5.000	1.216	1.130	46.0	49.3	53.1	53.1	0.6	0.6	0.489	0.343
0.7	1.395	4.138	1.343	1.240	43.0	44.7	48.4	48.4	0.7	0.7	0.656	0.530
0.8	1.463	3.429	1.512	1.410	41.0	39.7	42.6	42.6	0.8	0.8	0.836	0.721
0.9	1.600	2.727	1.729	1.656	37.5	34.7	36.2	36.2	0.9	0.9	0.932	0.914
1.0	2.000	2.000	2.000	2.000	30.0	30.0	30.0	30.0	1.0	1.0	1.000	1.000
1.1			2.331	2.464		25.7	24.3				0.954	0.936
1.2			2.728	3.073		22.0	19.5				0.900	0.851
1.3			3.197	3.856		18.8	15.6				0.833	0.740
1.4			3.744	4.842		16.0	12.4				0.756	0.618
1.5			4.375	6.063		13.7	9.9				0.673	0.536
1.6			5.096	7.554		11.8	7.9				0.594	0.443
1.7			5.913	9.352		10.1	6.4				0.545	0.332
1.8			6.832	11.496		8.8	5.2				0.488	0.279
1.9			7.859	14.032		7.6	4.3				0.411	0.244

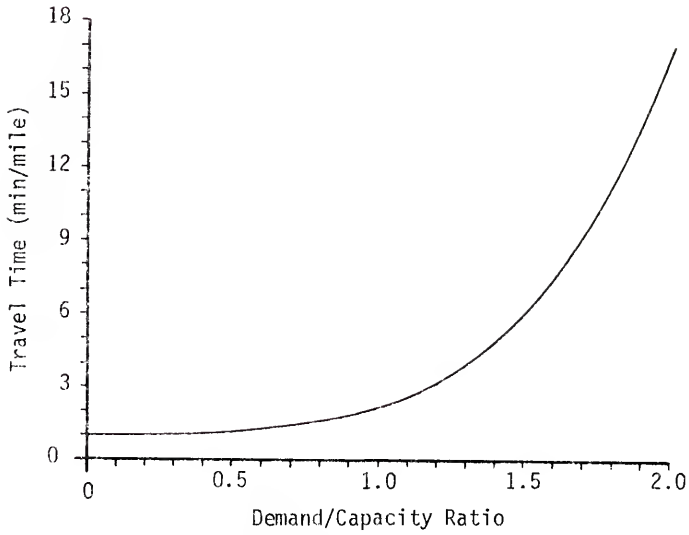


Figure A.4 EXTENDED TRAVEL TIME CURVE

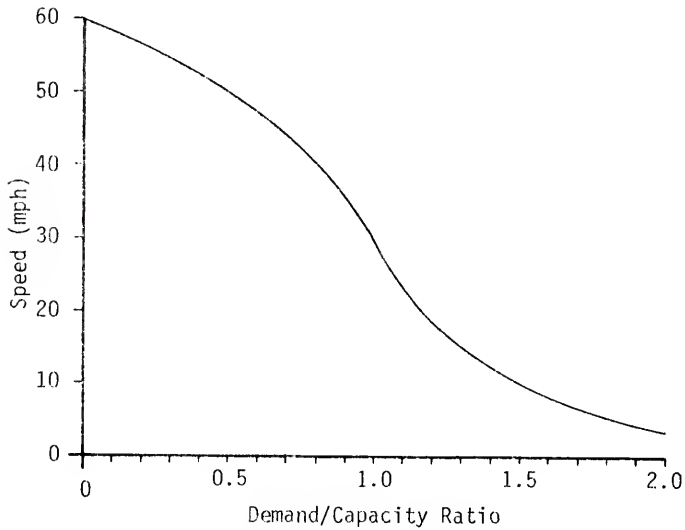


Figure A.5 EXTENDED DEMAND - SPEED RELATIONSHIP

APPENDIX B

SUMMARY OF THE I-95 CAR POOL DEFINITION ANALYSIS

Table B.1 RESULTS OF I-95 CAR POOL DEFINITION ANALYSES
FOR THE PERIOD 3:30 PM TO 6:30 PM

Section	Minimum Passenger Hours						Minimum Vehicle Hours					
	At Optimality		At Car Pool Definition Of				At Optimality		At Car Pool Definition Of			
	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv	Car-Pool	Veh-Hrs	2 ppv	3 ppv	4 ppv	5 ppv
1	2.8	79.9	79.9*	82.6			2.8	56.0	56.0*	58.0		
2	3.7	155.3	(154.5)	154.5*	156.5		3.7	(108.8)	(108.8)	108.2*	109.6	
3	3.3	423.6	(424.5)	424.5	430.9		3.0	299.0	299.0*	299.0		
4	2.9	124.0	125.1*	128.4			2.9	87.6	87.6*	91.4		
5	2.8	317.0	320.6*	344.2			2.8	224.5	224.5*	247.0		
6	2.8	505.6	509.0*	602.8			2.8	357.1	356.5*	436.5		
7	2.8	355.3	359.5*	388.1			2.8	251.7	251.7*	279.6		
8	2.8	875.7	900.0*	979.0			2.8	630.3	630.3*	715.2		
9	2.8	385.3	388.1*	446.2			2.8	274.3	271.8*	324.7		
10	2.8	308.0	317.1*	342.3			2.8	222.1	222.1*	250.2		
11	2.7	334.8	333.7*	390.3			2.8	237.8	233.7*	283.9		
12	2.8	1024.2	1049.5*	1156.4			2.8	734.9	734.9*	844.0		
13	2.8	377.2	382.7*	446.8			2.8	268.1	268.1*	323.1		
14	2.8	225.3	223.4*	240.4			2.8	158.6	156.5*	171.8		
15	2.8	332.1	336.6*	372.8			2.8	235.7	235.7*	268.3		
16	2.5	399.5	413.2*	549.7			2.5	289.4	239.4*	406.6		
17	2.6	957.1	995.9*	1241.1			2.5	697.4	697.4*	921.1		
18	2.5	579.9	604.8*	782.1			2.5	423.5	423.5*	579.4		
Tot.	NA	7759.8	7917.9	9072.7	--		NA	5556.8	5547.5	6607.0	--	

*Degree of priority ≤ 1.0

Parentheses indicate estimated values

Table B.2 RESULTS OF I-95 CAR POOL DEFINITION ANALYSES
FOR THE PERIOD 3:30 PM TO 4:00 PM

Section	Minimum Passenger Hours						Minimum Vehicle Hours					
	At Optimality		At Car Pool Definition Of				At Optimality		At Car Pool Definition Of			
	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv	Car-Pool	Veh-Hrs	2 ppv	3 ppv	4 ppv	5 ppv
1	5.8	12.0				12.0	5.6	8.4				8.4
2	3.0	23.2				23.2	3.7	16.2		16.2*	16.2	
3	2.9	59.5	59.7*	59.7			3.0	41.8	41.8*	42.0		
4	3.0	17.6	17.8*	17.7			2.9	12.5	12.5*	12.5		
5	2.8	41.5	42.1*	41.9			2.8	29.4	29.4*	29.7		
6	3.0	62.5	62.4*	66.5			2.8	43.8	43.6*	47.5		
7	2.8	46.4	47.0*	46.9			2.8	32.9	32.9*	33.4		
8	2.8	92.2	93.6*	103.7			2.7	65.4	65.4*	74.6		
9	2.8	43.9	44.0*	46.6			2.7	30.9	30.7*	33.3		
10	2.8	32.0	32.6*	36.6			2.7	22.8	22.8*	26.4		
11	2.8	39.0	38.9*	41.5			2.8	27.5	27.2*	29.7		
12	2.8	109.8	111.3*	120.4			2.8	77.8	77.8*	86.5		
13	2.8	47.3	47.4*	50.0			2.8	33.2	33.2*	35.6		
14	3.0	30.9	31.4*	31.0			3.0	22.0	22.0*	22.0		
15	2.9	42.9	43.3*	44.0			2.8	30.3	30.3*	31.3		
16	2.5	42.8	43.6*	60.8			2.4	30.5	30.5*	44.6		
17	2.4	100.3	100.6*	141.1			2.4	71.5	70.3*	104.1		
18	2.5	59.9	61.1*	87.3			2.4	42.8	42.8*	64.2		
Tot.	NA	903.7	--	--		--	NA	639.7	--	--	--	--

*Degree of Priority ≤ 1.0

Table B.3 RESULTS OF I-95 CAR POOL DEFINITION ANALYSES
FOR THE PERIOD 4:00 PM TO 4:30 PM

Section	Minimum Passenger Hours						Minimum Vehicle Hours					
	At Optimality		At Car Pool Definition Of				At Optimality		At Car Pool Definition Of			
	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv	Car-Pool	Veh-Hrs	2 ppv	3 ppv	4 ppv	5 ppv
1	2.5	16.4	16.5*	17.8			2.5	11.5	11.5*	12.5		
2	4.3	33.7			33.7	34.2	3.4	23.5	23.5	23.5		
3	4.3	90.6			90.5	94.0	2.9	63.4	63.4*	63.4		
4	2.7	33.3	33.0*	41.1			2.7	23.4	23.0*	29.5		
5	2.8	102.6	105.9*	119.6			2.3	73.6	73.6*	86.5		
6	2.8	174.1	177.3*	196.2			2.7	123.9	123.3*	142.4		
7	2.8	115.2	119.0*	134.3			2.8	82.7	82.7*	97.2		
8	2.9	278.9	287.4*	297.8			2.7	199.7	199.7*	216.6		
9	2.8	127.9	131.1*	140.2			2.7	91.1	91.1*	101.8		
10	2.9	97.4	100.4*	103.5			2.7	59.8	59.8*	75.3		
11	2.8	112.3	114.9*	124.1			2.7	79.9	79.9*	90.1		
12	2.9	329.7	339.4*	354.3			2.7	235.9	235.9*	257.8		
13	2.8	129.4	131.8*	146.9			2.7	92.0	91.6*	106.5		
14	2.7	68.2	69.8*	80.6			2.7	48.5	48.5*	58.1		
15	2.8	109.1	112.9*	126.9			2.8	78.5	78.5*	91.9		
16	2.7	127.9	133.2*	156.3			2.5	92.6	92.6*	114.3		
17	2.7	287.1	300.8*	333.0			2.5	209.0	209.0*	243.8		
18	2.7	181.7	189.7*	217.3			2.5	131.8	131.8*	159.0		
Tot.	NA	2415.5	--	--	--	--	NA	1730.8	1729.4	1970.2		

*Degree of Priority ≤ 1.0

Table B.4 RESULTS OF I-95 CAR POOL DEFINITION ANALYSES
FOR THE PERIOD 4:30 PM TO 5:00 PM

Section	Minimum Passenger Hours						Minimum Vehicle Hours					
	At Optimality		At Car Pool Definition Of		At Optimality		At Car Pool Definition Of		At Optimality		At Car Pool Definition Of	
	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv
1	2.4	15.8	15.9*	16.9			2.4	11.3	11.3*	12.2		
2	4.1	32.4	32.4	32.8			3.7	23.2	23.2*	23.2		
3	4.1	87.2			87.2	89.0	3.0	62.7	62.7*	62.7		
4	2.7	31.1	30.7*	36.4			2.7	22.4	21.9*	27.0		
5	2.8	96.2	99.7*	106.0			2.8	71.1	71.1*	79.3		
6	2.8	162.9	167.1*	175.4			2.7	119.5	119.1*	131.7		
7	2.8	108.1	112.1*	119.1			2.8	79.9	79.9*	89.1		
8	2.9	262.2	272.2*	269.7			2.7	194.0	194.0*	203.4		
9	2.9	119.9	123.7*	125.9			2.7	88.2	88.2*	94.9		
10	2.9	91.6	95.2*	94.0			2.7	67.8	67.8*	70.9		
11	2.9	105.2	103.3*	111.3			2.7	77.3	77.2*	83.8		
12	2.9	309.7	321.1*	320.0			2.7	228.9	228.9*	241.4		
13	2.8	120.9	124.6*	131.0			2.7	88.8	88.8*	98.4		
14	2.7	63.8	65.4*	72.1			2.7	46.6	46.6*	53.7		
15	2.8	102.4	106.4*	112.7			2.8	75.9	75.9*	84.4		
16	2.6	120.0	126.0*	141.8			2.4	89.9	89.9*	107.9		
17	2.7	269.9	285.6*	303.5			2.4	203.7	203.7*	231.4		
18	2.6	170.5	179.8*	197.4			2.4	128.1	128.1*	150.3		
Tot.	NA	2269.8	--	--	--	--	NA	1679.3	1678.3	1845.7		

*Degree of Priority ≤ 1.0

Table B.5 RESULTS OF 1-95 CAR POOL DEFINITION ANALYSES
FOR THE PERIOD 5:00 PM TO 5:30 PM

Section	Minimum Passenger Hours						Minimum Vehicle Hours					
	At Optimality		At Car Pool Definition Of				At Optimality		At Car Pool Definition			
	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv	Car-Pool	Veh-Hrs	2 ppv	3 ppv	4 ppv	5 ppv
1	2.5	14.2	14.2*	15.2			2.5	10.0	10.0*	10.8		
2	3.4	28.6		28.7*	29.0		3.4	20.2	20.2*	20.5		
3	3.4	77.9		78.1	79.2		2.9	55.3	55.3*	55.3		
4	2.7	24.0	24.0*	25.8			2.7	17.0	16.9*	18.5		
5	2.7	66.8	66.0*	80.4			2.7	47.5	46.5*	58.4		
6	2.8	116.0	118.6*	134.1			2.8	83.5	83.5*	97.7		
7	2.7	75.1	74.5*	90.4			2.7	53.5	52.4*	65.6		
8	2.8	193.1	197.4*	214.8			2.7	138.9	138.9*	157.4		
9	2.8	86.4	88.9*	98.0			2.8	62.6	62.6*	71.6		
10	2.8	67.9	69.1*	75.0			2.7	48.8	48.6*	55.0		
11	2.8	75.5	77.6*	85.9			2.8	54.6	54.6*	62.7		
12	2.8	225.8	232.4*	253.7			2.8	163.5	163.5*	185.8		
13	2.7	85.9	87.5*	100.0			2.7	61.6	61.6*	72.8		
14	2.8	44.8	45.4*	53.1			2.7	32.0	32.0*	38.3		
15	2.7	71.9	72.1*	85.7			2.7	51.4	50.7*	62.3		
16	2.6	88.3	91.3*	116.8			2.5	64.3	64.3*	86.3		
17	2.6	208.9	216.3*	256.2			2.5	152.1	152.1*	189.4		
18	2.5	128.0	131.3*	164.4			2.5	92.9	92.4*	121.4		
Tot.	NA	1679.1	--	1956.3	--		NA	1209.7	1206.1	1429.8		

*Degree of Priority ≤ 1.0

Table B.6 RESULTS OF I-95 CAR POOL DEFINITION ANALYSES
FOR THE PERIOD 5:30 PM TO 6:00 PM

Section	Minimum Passenger Hours						Minimum Vehicle Hours					
	At Optimality		At Car Pool Definition Of				At Optimality		At Car Pool Definition Of			
	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv	Car-Pool	Veh-Hrs	2 ppv	3 ppv	4 ppv	5 ppv
1	4.8	12.5			12.5	12.6	4.8	8.5			8.5	8.5
2	6.0	24.2				24.2	3.9	16.4		16.4*	16.4	
3	3.0	62.5		62.5	63.9		3.0	42.5		42.5*	43.5	
4	3.0	18.4	18.7*	18.4			3.0	12.6	12.6*	12.7		
5	3.0	43.6	44.1*	44.1			2.8	29.8	29.8*	30.3		
6	2.8	66.3	65.5*	70.0			2.8	45.1	44.3*	48.3		
7	3.0	48.8	49.3*	49.5			2.8	33.3	33.3*	34.0		
8	2.8	97.8	99.4*	109.6			2.8	67.2	67.3*	76.4		
9	2.8	46.5	46.8*	49.0			2.8	31.8	31.7*	33.9		
10	2.8	34.0	34.6*	38.7			2.8	23.4	23.4*	27.0		
11	2.8	41.4	41.4*	43.6			2.8	28.2	28.0*	30.2		
12	2.8	116.6	118.4*	127.5			2.8	80.1	80.1*	88.7		
13	2.8	49.9	49.7*	52.6			2.8	33.9	33.6*	36.3		
14	3.2	32.5		32.5	33.4		3.2	22.3		22.3	22.8	
15	2.8	45.0	45.3*	46.3			2.8	30.7	30.7*	31.9		
16	2.6	45.5	46.4*	63.7			2.5	31.4	31.4*	45.3		
17	2.5	108.4	109.8*	147.7			2.5	75.0	74.4*	105.8		
18	2.5	64.2	64.9*	91.3			2.5	44.0	44.0*	65.1		
Tot.		958.1	--	--	--	--	NA	656.3	--	--	--	--

*Degree of Priority ≤ 1.0

Table B.7 RESULTS OF I-95 CAR POOL DEFINITION ANALYSES
FOR THE PERIOD 6:00 PM TO 6:30 PM

Section	Minimum Passenger Hours					Minimum Vehicle Hours								
	At Optimality		At Car Pool			Definition Of		At Optimality		At Car Pool			Definition Of	
	Car-Pool	Pax-Hrs	2 ppv	3 ppv	4 ppv	5 ppv	Car-Pool	Veh-Hrs	2 ppv	3 ppv	4 ppv	5 ppv		
1	6.0	10.1				10.1	6.0	7.4					7.4	
2	6.0	19.3				19.3	3.3	14.3			14.3			
3	4.8	47.4			47.4	47.5	2.9	35.2	35.2*					
4	2.7	13.8	13.9*	14.3			2.7	10.2	10.2*	10.6				
5	2.8	32.7	33.1*	33.5			2.8	24.3	24.3*	24.9				
6	2.9	48.7	49.3*	49.6			2.9	36.3	36.3*	36.9				
7	2.8	36.6	37.0*	37.4			2.8	27.2	27.2*	27.8				
8	2.9	65.5	66.3*	68.4			2.7	48.7	48.7*	51.0				
9	3.0	32.8	33.2*	33.2			2.7	24.4	24.4*	24.7				
10	2.8	22.6	22.8*	23.8			2.8	16.8	16.8*	17.8				
11	2.9	29.5	29.9*	29.9			2.7	22.0	22.0*	22.3				
12	2.9	79.0	80.1*	81.5			2.9	58.9	58.9*	60.7				
13	2.9	37.0	37.5*	37.7			2.7	27.5	27.5*	28.0				
14	2.8	24.4	24.6*	25.1			2.8	18.1	18.1*	18.6				
15	2.8	33.7	34.1*	34.5			2.7	25.1	25.1*	25.6				
16	2.6	30.8	31.2*	36.3			2.3	23.0	23.0*	27.2*				
17	2.3	63.2	62.3*	87.8			2.3	46.8	45.9*	66.5*				
18	2.4	42.0	42.4*	51.5			2.3	31.2	31.2*	38.7				
Tot.	NA	669.1	--	--	--	--	NA	497.4	--	--	--	--	--	

* Degree of Priority ≤ 1.0

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BIOGRAPHICAL SKETCH

Thomas Hamilton Culpepper was born in Marianna, Florida, on November 24, 1949. He attended public schools in Chipley, Florida, and was graduated from Washington County High School in 1967.

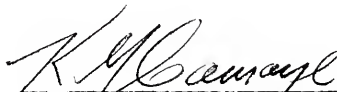
In September, 1967 he enrolled in Chipola Junior College, and, after completion of a two-year program in the liberal arts, transferred to the University of Florida. He was awarded the Bachelor of Science in Civil Engineering degree in August, 1972.

He entered the Graduate School at the University of Florida in September, 1972, and subsequently received the Master of Engineering degree in December, 1973. Since that time, Mr. Culpepper has continued work toward the Doctor of Philosophy degree with major in Civil Engineering and emphasis in Transportation.

During the course of his academic training, he has held appointments as Graduate Research Assistant and Graduate Research Associate in the Department of Civil Engineering, where he has been actively engaged in the research program of the Transportation Research Center.

Thomas H. Culpepper is married to Maryanne Terese Gillis and has one daughter, Katharine. He is a member of the Transportation Research Board, the Institute of Transportation Engineers, and the National Society of Professional Engineers.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.



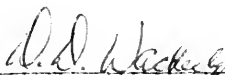
K. G. Courage, Chairman
Associate Professor of Civil Engineering

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J. A. Wattlesworth
Professor of Civil Engineering

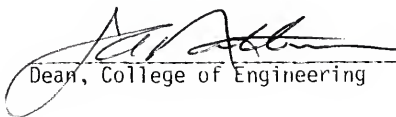
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This dissertation was submitted to the Graduate Faculty of the College of Engineering and to the Graduate Council, and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy.

August 1977



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